Chapter Five

Superstructure Design

5.1 INTRODUCTION

This chapter addresses superstructure design for construction of new structures. For superstructure design on rehabilitation and restoration projects, see Chapter Nine.

5.2 DESIGN CRITERIA

5.2.1 BRIDGE AND APPROACH GEOMETRY

5.2.1.1 Design Standards

For roadway approach geometric design, follow the geometric standards in the Department's *Road Design Manual*. Where additional information is needed, refer to AASHTO's *Green Book.*

For bridge design, follow the procedures in the *AASHTO Specifications* and this manual. Refer to Chapter Four of this manual for rating and evaluation procedures.

5.2.1.2 Length

The length of the bridge will be decided at the type, size and location (TS&L) stage of plan development, or at the preliminary plan stage when TS&L plans are not required. The length of a bridge is measured along the centerline of the roadway from the back of the abutment to the back of abutment. Generally, several alternates are developed considering various materials. Cost comparisons are performed to identify the most economical alternative. As outlined in Chapter Two, life cycle cost is one criterion used in developing an evaluation matrix.

Considerations in determining the lengths of bridges over roadways include:

- roadway, shoulder, and median widths,
- future widening of roadways under or over the structure,
- section openness and visibility,
- span length uniformity and continuous span requirements,
- economics,
- safety,
- aesthetics, and
- drainage.

The structure must be long enough to allow for full-width shoulders on the existing roadway being crossed. As a minimum, the Department policy is to continue the same shoulder width through the structure as on the existing roadway. The designer must verify that all clearances and bridge widths result in a structure that is not functionally obsolete.

The length of structures over water will be determined from the hydraulic analysis and environmental considerations. Refer to Chapter Three.

5.2.1.3 Width

The bridge width includes the roadway, shoulders and offset for the barrier rail. The roadway and shoulders should match the existing approach roadway width design standards. Follow the design standards and controls in Chapters Two and Three of the *Road Design Manual*. These design criteria include levels of service, roadway classification, design speed, traffic volumes, traffic composition, and traffic projections.

The designer should consider the need for future widening. The impact on traffic and construction operations at the time the structure will be widened must also be considered.

The longitudinal joint in the deck should not be in a wheel path after the deck is widened.

Outside shoulder widths may be narrowed on long bridges to reduce costs. A long bridge is defined as one longer than 200 feet [60 m]; see Chapter 4 of the *Green Book* for the criteria. Narrowing or eliminating shoulders below AASHTO standards requires a design exception approved by the Chief Engineer.

For the offset between the outside edge of the shoulder and the face of the barrier, two feet [0.6 m] should be allowed. This distance may be modified with a design exception.

5.2.1.4 Skew

Skew is defined as the acute angle subtended by a line perpendicular to the longitudinal axis of the structure and the centerline of bearings. The designer should minimize the skew angle of the substructure and the superstructure for simplicity of design and construction. Reducing the skew also improves overall performance of the structure. To minimize skew, longer spans or the use of a curved girder design may be considered where applicable.

The designer must perform a more accurate analysis, such as finite element, for highly skewed bridges (those with skews of 60 degrees or greater).

5.2.1.5 Clearances

5.2.1.5.1 Vertical Clearance

Vertical clearance is checked by determining the lowest vertical distance between points on the roadway and the corresponding points on the bridge superstructure within the traveled way.

The minimum vertical clearance for roads over Interstate, U.S. and Delaware routes is 16'-6" [5 m]. The minimum clearance over local roads is 14'-6" [4.4 m]. Pedestrian bridges and overhead sign bridges must have 17'-6" [5.3 m] clearance for all roads. These clearances include 6 inches [150 mm] for future resurfacing.

When determining the minimum clearance, the future widening must be considered.

5.2.1.5.2 Horizontal Clearance and Clear Zone

See Chapter Three of the *Road Design Manual* for discussion on horizontal clearance and clear zone for roadways and bridge approaches. Desired clear zone widths cannot normally be attained for roadways under bridges at reasonable costs. Where the desired clear zone cannot be attained, protection such as guardrails or rigid barriers is provided.

5.2.1.5.3 Railroad Clearance

The minimum clearances for railroads must be mutually established between the Department and each railroad and meet applicable State laws or regulations. Anticipated future tracks and track adjustments should be considered in establishing the clearances. Sight distances to railroad signals should not be reduced by construction of the new bridge. The construction requirements, including such items as forms and protection shields, must be considered in meeting the temporary clearance criteria. Railroad tracks must be protected from damage and debris during demolition, construction, and deck replacement. A protection shield must be erected over the track to catch debris. Shields are designed to meet case-by-case needs. Temporary construction clearances may be less than standard clearance requirements, if approved by the railroad.

The minimum vertical clearance above the top of the highest rail is 23'-0" [7.01 m]. Consider the profile grade of the rail and the configuration of the bridge to check the minimum clearance. In areas where a railroad has been electrified with a catenary wire, and areas likely to be electrified, the minimum vertical clearance must be 24'-6" [7.47 m] above the highest rail.

The minimum horizontal clearance measured from the centerline of track to the near face of the obstruction must be 20'-0" [6.1 m] for tangent track and 21'-0" [6.4 m] for curves. On curved tracks, horizontal clearances should be increased 1.5 inches [38 mm] per degree of curvature. When the track is superelevated, clearances on the inside of the curve must be increased by 3.5 inches [88 mm] per 1 inch [25 mm] of superelevation. The desired horizontal clearance is 25'-0" [7.62 m]. All piers less than 25'-0" [7.62 m] from the centerline of the track require a crash wall designed in accordance with the American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering*. Refer to Figure 5-1 for crash wall details.

The designer must consider the railroad's drainage and utility system in determining horizontal clearances. Piers and end slopes shall be located such that they do not interfere with railroad drainage ditches.

See Figure 5-2 for requirements.

Figure 5-1
Crashwall Details

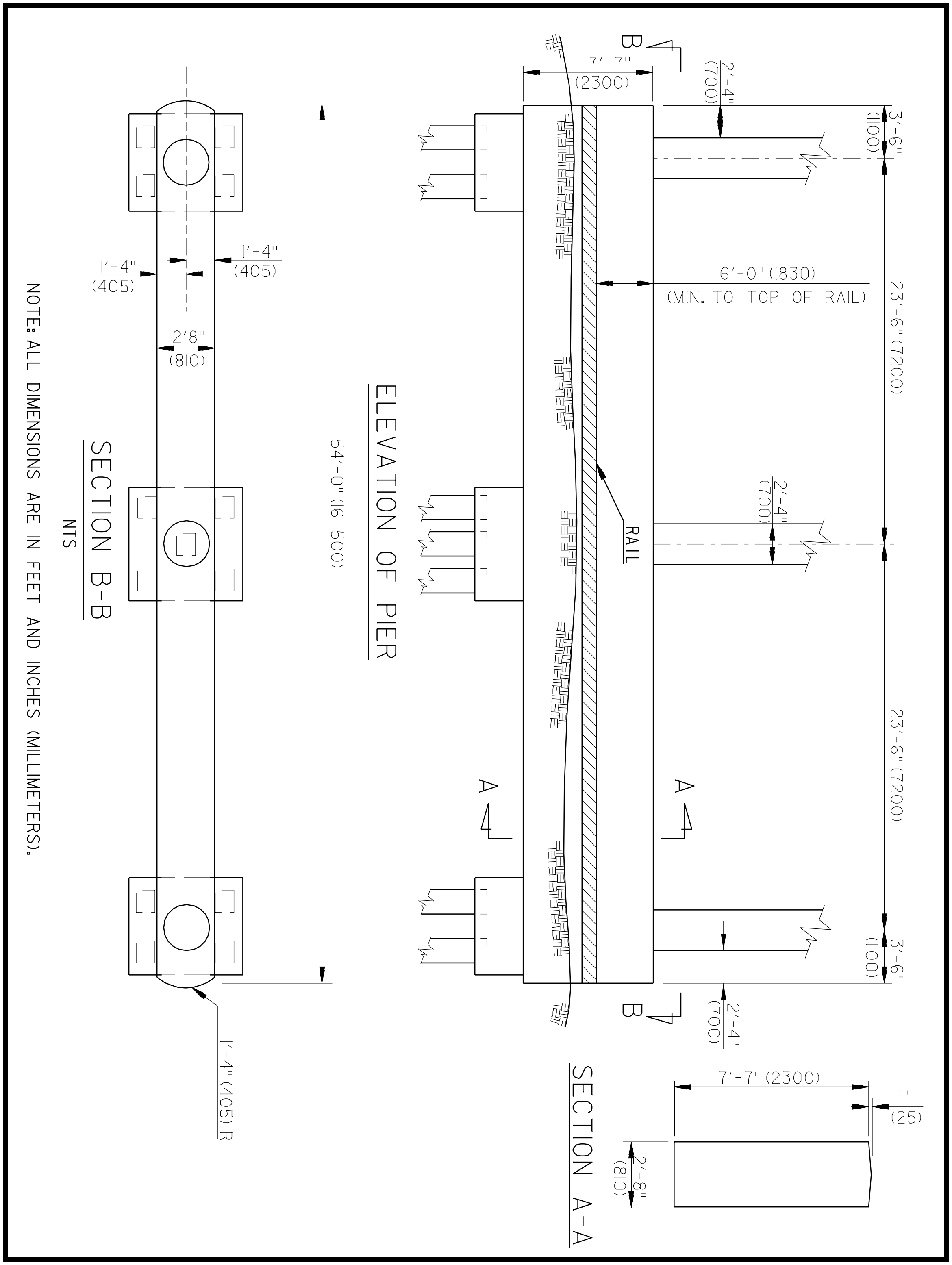
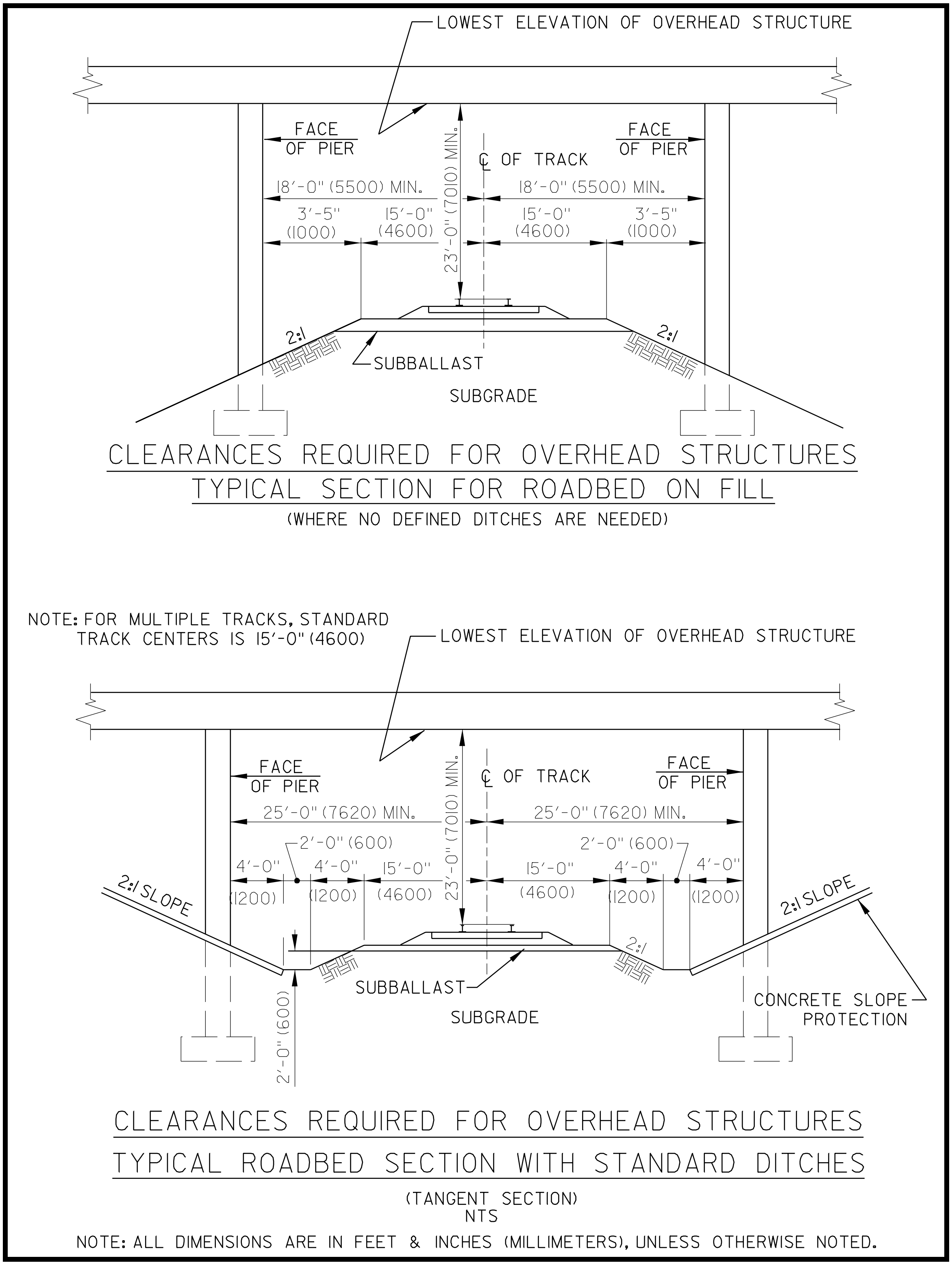


Figure 5-2
Railroad Clearances



5.2.2 STRUCTURAL ANALYSIS

5.2.2.1 AASHTO Method

The Department follows the approximate methods given in Section 4, Structural Analysis and Evaluation, of the *AASHTO Specifications*.

5.2.2.2 Refined Method

On a case-by-case basis, the designer may use another method, such as the finite element analysis method for highly skewed bridges or complex structures, to refine the load distribution. The approval of the Bridge Design Engineer is required.

5.2.3 DESIGN LOADS

5.2.3.1 Live Loads and Lane Loads

Live loads and lane loads used for design shall comply with the AASHTO loads as defined in Chapter Two of this manual. Application of live loads, including vehicles and pedestrians, will be in accordance with the *AASHTO Specifications*.

5.2.3.2 Dead Loads

The Department follows the *AASHTO Specifications* for dead loads. Dead loads consist of the weight of the entire superstructure including beams, deck, sidewalks, tracks, parapets, pipes, conduits, cables and other public utility services. Unless future additional utilities are anticipated, they will not be considered. The design must include an allowance for an additional future 2 inch [50 mm] overlay and an additional 0.5 inch [13 mm] integral wearing surface. When a design includes stay-in-place deck forms, their weight must be included as part of the dead load. Use the following loads:

- Future overlay: 25 psf [1.2 kN/m^2]
- Stay-in-place forms (including concrete-in-form corrugations): 15 psf [0.7 kN/m^2]
- Reinforced concrete: 150 pcf [23.6 kN/m^3]
- Lightweight concrete: 110 to 130 pcf [17.3 to 20.4 kN/m^3]
- Structural steel: 490 pcf [77.0 kN/m^3]
- Fill soil: 120 pcf [18.8 kN/m^3]
- Water: 62.4 pcf [9.8 kN/m^3]
- Timber: Softwood 50 pcf [7.9 kN/m^3]; Hardwood 60 pcf [9.5 kN/m^3]
- Asphalt: 140 pcf [22.0 kN/m^3]
- Stone masonry: 170 pcf [26.7 kN/m^3]
- Brick: 140 pcf [22.0 kN/m^3]

5.2.3.3 Dynamic Load Allowance

For dynamic load allowance, refer to Section 3.6.2, Dynamic Load Allowance: IM, *AASHTO Specifications*.

5.2.3.4 Combinations of Loads

See Section 3.4, Load Factors and Combinations, *AASHTO Specifications.*

5.2.4 DESIGN METHODS

The Department uses the load and resistance factor method of design as defined in the *AASHTO Specifications.*

5.2.5 TEMPERATURE RANGE

Bridge designs must allow for movements due to temperature. Both steel and concrete structures expand and contract because of temperature changes, but the rate of change for massive concrete members or structures is slower than for steel. Use a temperature range of 0 to 120 ºF [-18 to +49 ºC] for steel bridges, and 10 to 80 ºF [-12 to +27 ºC] for concrete bridges. Refer to Section 3.12, Force Effects due to Superimposed

Deformations: TU, TG, SH, CR, SE, in the *AASHTO Specifications*.

5.2.6 DEFLECTIONS

The Department adopts the deflection criteria given in Section 2.5.2.6.2, Criteria for Deflection, *AASHTO Specifications*.

5.3 CONCRETE BRIDGE DECKS

This section applies to the design of bridge decks supported by steel, concrete or timber beams.

5.3.1 DESIGN OF CAST-IN-PLACE DECKS

5.3.1.1 Materials

Class D portland cement concrete (f'_c = 4,500 psi [30 MPa] at 28 days) is used for concrete decks.

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified.

All reinforcing steel shall be protected with fusion-bonded epoxy. Epoxy coating conforming to AASHTO M284 [M284M] shall be specified.

5.3.1.2 Design

In applying wheel loads when designing the deck, refer to Section 4.6.2.1, Decks, in the *AASHTO Specifications*. For design, refer to Section 9, Decks and Deck Systems.

The design of the overhang shall also conform to Section 4.6.2.1, Decks, in the *AASHTO Specifications* and meet the following criteria:

- normal overhang is 2'-6" [0.75 m], and
- maximum overhang is half the beam spacing or 4'-0" [1.2 m], whichever is less.

The designer must check the constructibility of the overhang.

5.3.1.3 Thickness

The minimum thickness of concrete decks is 8 in [210 mm]. The maximum is 10 in [250 mm]. In general, the thickness is measured above the top of the stay-in-place forms. The total thickness includes 0.5 in [15 mm] for an integral wearing surface. Therefore the minimum design thickness is 7.5 in [190 mm]. The wearing surface is not considered a part of the design thickness.

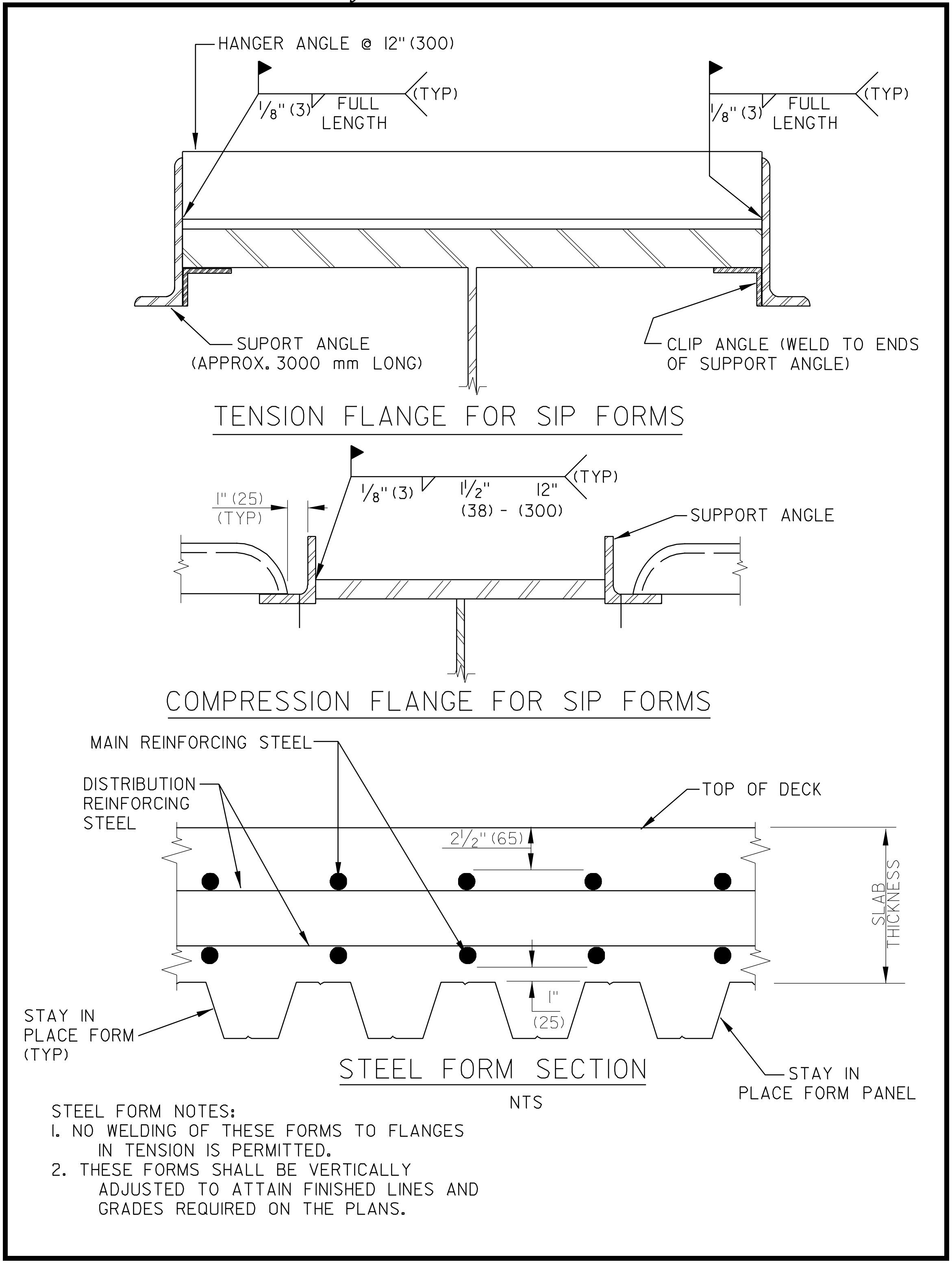
Where corrugated metal stay-in-place forms are used, the thickness is measured to the top of the corrugation as shown in Figure 5-3.

5.3.1.4 Concrete Cover

The minimum cover over reinforcing bars is:

- top mat - 2.5 in [65 mm], which includes the 0.5 in [15 mm] integral wearing surface, and
- bottom mat – When stay-in-place forms are used, the minimum cover shall be 1 in [25 mm]; otherwise it shall be 2 in [50 mm], unless approved by the Bridge Design Engineer.

Figure 5-3
Stay-in-Place Form Details



5.3.1.5 Reinforcement

The minimum size of reinforcing bar is #5 [16]. Because of transportation problems, the longest reinforcing bars used in Delaware are 60 feet [18.3 m]. Specify long bars, insofar as possible, to minimize splicing. Refer to Section 5.11.5, Splices of Bar Reinforcement, *AASHTO Specifications,* for splicing requirements. Normally, reinforcing steel splices are lapped, but the designer may specify mechanical splices. Where mechanical splices are not included on the plans, they may be approved by the Department if requested by the contractor. Welded splices are not permitted.

5.3.1.6 Stay-in-Place Forms

Stay-in-place (SIP) forms must conform with the requirements for permanent steel bridge deck forms in Section 602 of the *Standard Specifications*. See Figure 5-3 for details. Welding forms to structural components is not permitted in tension zones. SIP forms are preferred by DelDOT. If they should not be used, then it should be noted on the plans.

5.3.1.7 Pouring Decks

The superstructure design should be evaluated to determine the need for a pouring sequence. The designer should consider the effect of the plastic concrete on the girders in evaluating the need for a pouring sequence. The designer must consider beam or girder deflection in developing the pouring sequence. Refer to Section 602 of the *Standard Specifications* for the construction method. Normally, deck concrete placement begins at the low point on the deck and proceeds up grade. Where the deck grade is minor, this requirement may be waived. Bridge deck pouring sequences must be submitted for review and shown on the plans, as appropriate. The number of pours should be minimized for ease and speed of construction.

The bridge designer is expected to attend the “pre-pour” conference with the contractor and other representatives of the Department. Refer to Chapter Thirteen for additional details. The contractor may be permitted to pour the deck continuously with approval of the Bridge Design Engineer. Any deviation from the pouring sequence should be reviewed and approved by the designer.

5.3.2 DESIGN OF PRECAST SLABS AND DECK PANELS

This section applies to the design of precast and precast-prestressed deck slabs and deck panels supported by steel or concrete beams.

Precast deck slabs are cast full thickness in segments for placement on the beams. They may be conventionally reinforced or prestressed. A concrete wearing surface shall be placed to improve the ride.

Alternatively, precast deck panels are cast partial thickness and are placed to act as stay-in-place forms. The remainder of the deck is cast-in-place to form a full thickness composite deck. They may be conventionally reinforced or prestressed.

If deck slabs and deck panels are prestressed, typically the prestressing will be in the transverse direction. Deck slabs and panels may be longitudinally post-tensioned prior to pouring the deck or wearing surface. Hot-mix asphalt overlays are only permitted for temporary applications on precast slabs or deck panels. Precast slabs and deck panels may be utilized on a case-by-case basis. The advantages of precast slabs and deck panels include:

- expediting construction; and
- reducing traffic delays and inconvenience to the traveling public.

These advantages must be evaluated against the disadvantages that include:

- the cost of precast concrete is higher than for CIP concrete;
- increased potential for cracking of the deck at slab and panel joints; and
- design details are more critical to fabrication and construction.

Proper support of deck panels on the beams is critical to avoid deck cracking. Proper tolerance should be enforced at the shop to minimize the gap at the joints.

5.3.2.1 Materials

Class D portland cement concrete (f'_c = 4,500 psi [30 MPa] at 28 days) is used for non-prestressed deck slabs. The strength of concrete needed to meet the design requirements should be specified for prestressed deck slabs and deck panels. If the design strength is higher than 4,500 psi [30 MPa], a special mix design is required.

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified. The minimum size of reinforcing is a #5 [16] bar.

All reinforcing steel shall be protected with fusion-bonded epoxy. Epoxy coating conforming to AASHTO M284 [M284M] shall be specified.

Prestressing strands shall be a low relaxation strength specified by the designer.

5.3.2.2 Design

In applying the wheel loads when designing the slab, refer to Section 4.6.2.1, Decks, *AASHTO Specifications*. The designer must consider:

- application of live loads to the precast slab or deck panel as required for maintenance of traffic;
- construction requirements for handling and placing the slabs and panels;
- provisions for bonding the wearing surface to the slabs or panels;
- composite action between the precast panel and CIP reinforced concrete.

Refer to Section 5, Concrete Structures, and Section 9, Decks and Deck Systems, in the *AASHTO Specifications* for design of precast slabs and deck panels.

The design of the overhang shall also conform with Section 4.6.2.1, Decks, in the *AASHTO Specifications* and meet the following criteria:

- normal overhang is 2'-6" [0.75 m], and
- maximum overhang is half the beam spacing.

Figure 5-4 shows a typical overhang for a precast, prestressed deck panel.

Precast deck slabs and deck panels must be connected to the beams. Figure 5-5 shows typical connections for precast deck panels.

5.3.2.3 Thickness

The minimum thickness, with the overlay, for precast slabs is 8 inches [210 mm] and the maximum is 10 inches [250 mm]. A concrete overlay shall be applied to precast slabs to provide a wearing surface and improve the ride. Hot-mix asphalt overlays are only permitted for temporary applications. The minimum overlay thickness is 1.5 in [40 mm]. See Figure 5-6.

The minimum total thickness of the deck, including precast deck panels, cast-in-place composite deck, and integral wearing surface, is 8 inches [210 mm] and the maximum is 10 inches [250 mm]. The minimum thickness of the integral wearing surface is 0.5 inches [15 mm]. See Figure 5-7.

The wearing surface on precast slabs and precast panel decks is not considered a part of the design thickness for strength or resistance purposes.

Figure 5-4
Typical Precast Deck Slab

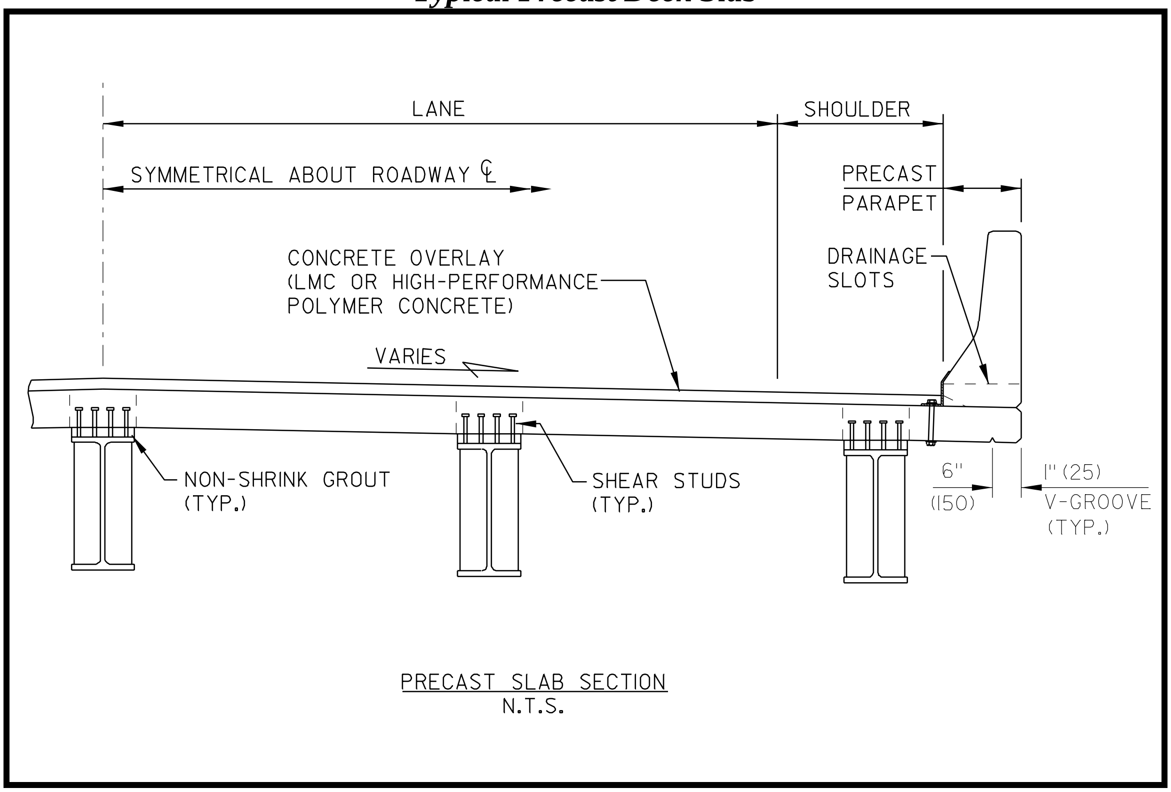


Figure 5-5
Typical Precast Panel to Beam Connections

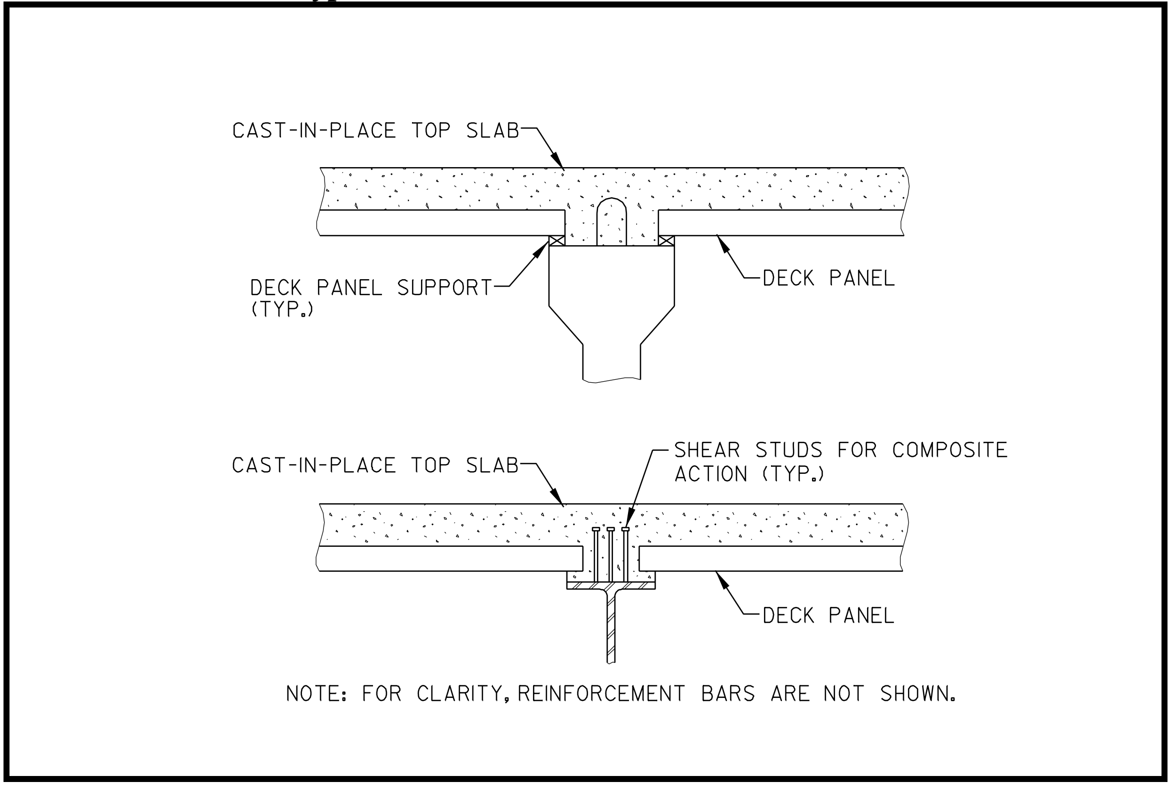


Figure 5-6
Dimensions of Precast Deck Slab

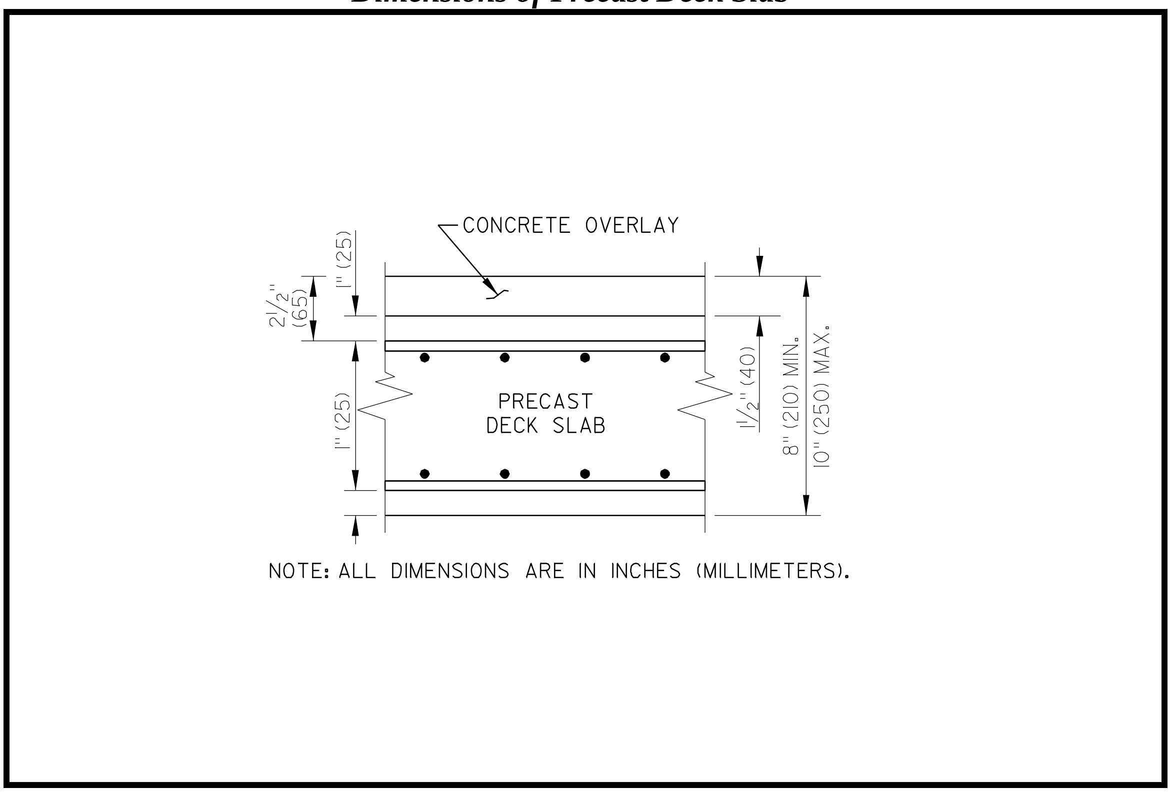
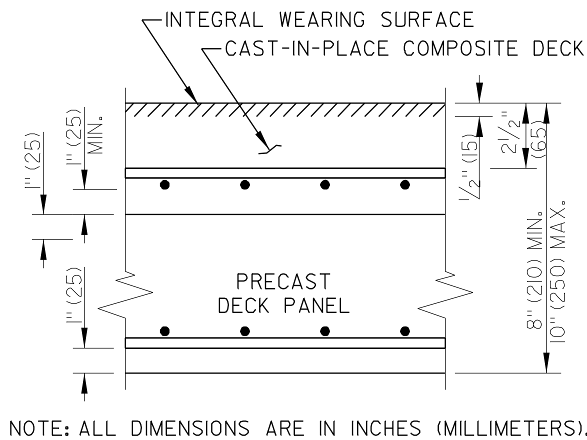


Figure 5-7
Dimensions of Precast Deck Panels



5.3.2.4 Concrete Cover

The minimum allowable cover over reinforcing bars in the precast slab or panel is:

- top mat—2.5 inches [65 mm] for precast slabs and 1 inch [25 mm] for precast panels with concrete overlay; and
- bottom mat—1 inch [25 mm] for either.

5.3.2.5 Reinforcement Splices

Splicing reinforcing bars will not be permitted in precast slabs or panels.

5.3.2.6 CIP Concrete

The superstructure design should be evaluated to determine the need for a pouring sequence. For details, see Secion 5.3.1.7.

5.3.3 DESIGN OF COMPOSITE ACTION DECKS

Composite action decks are designed such that both the deck and beam or girder respond to live loads and superimposed dead loads as a unit. Superimposed dead loads include all dead loads placed on the deck after it is cured. For steel beams, the interconnection is accomplished using studs attached to the top flange of the beam or girder. See Section 5.3.3.6. For concrete beams, the interconnection is accomplished using steel reinforcing bars embedded in the beam or girder and extending into the deck. See Section 5.4.3.3. The Department recommends the use of composite action when possible.

Refer to Section 9, Decks and Deck Systems, in the *AASHTO Specifications* for design parameters.

5.3.4 FINISHED DECK ELEVATIONS

The designer must include the framing plan and camber diagram for each span, as line drawings, in the plans. Finished deck elevations are shown in the plans, at the centerline of bearing over each abutment and pier line, and at 1/10th points or at 10 foot [3 m] intervals, whichever is less, the same as for camber diagrams:

- longitudinally over each beam;
- longitudinally along the span at the break points in the cross slope of the deck; and
- longitudinally at the face of the parapet.

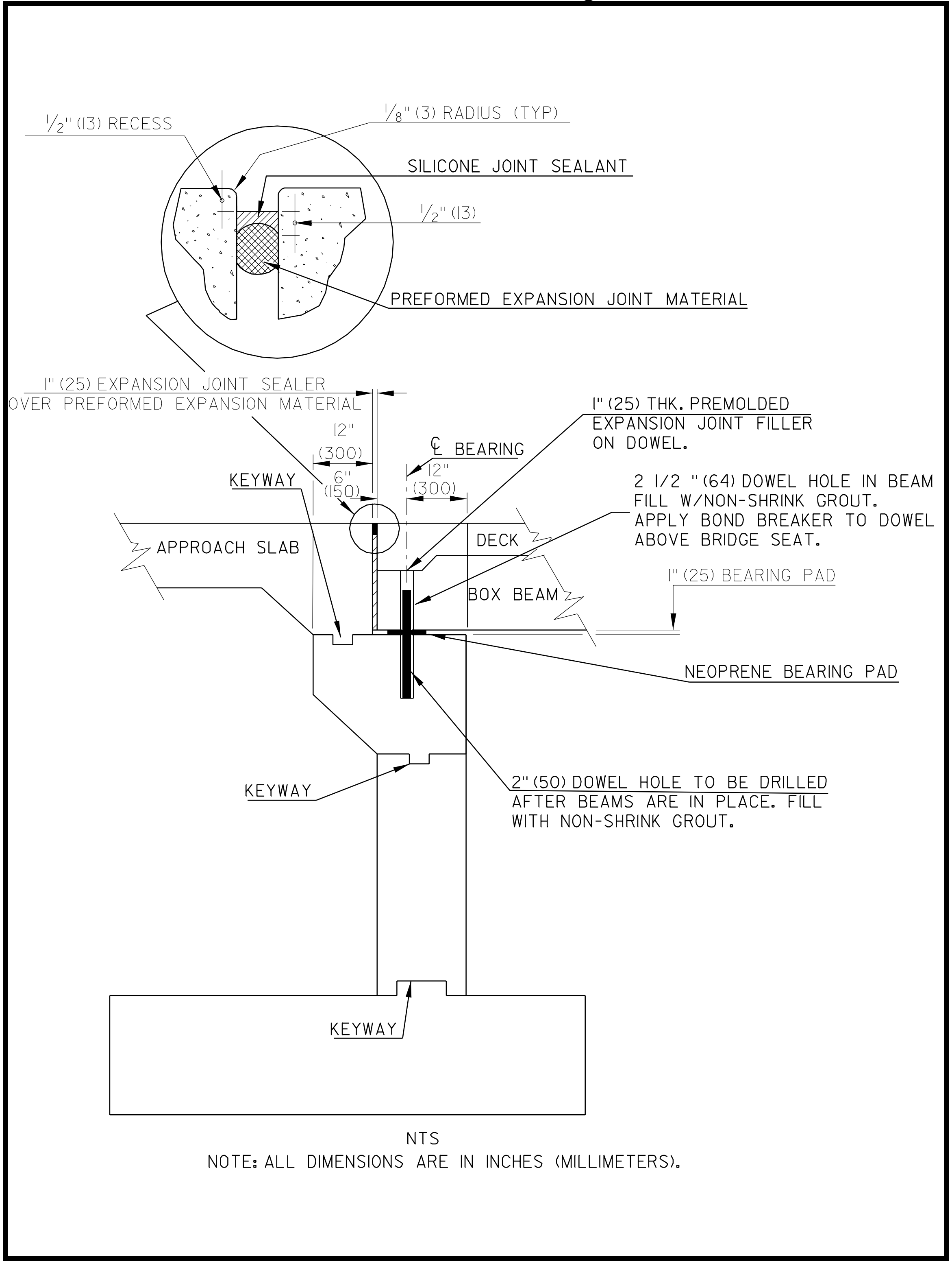
Refer to Section 5.3.4 for camber diagrams for steel beams and Section 5.4.3.7 for concrete beams.

5.3.5 JOINTS

Joints are constructed in decks to accommodate movement (rotation, deflection of the span, expansion and contraction). All joints must be sealed or constructed with troughs to prevent leakage of water onto the bearings and substructure.

Obtaining a watertight bridge joint is a difficult objective over the life of a bridge. Therefore, the design should incorporate the fewest number of joints that will provide for the necessary expansion and contraction. Jointless bridges should be designed whenever practical.

Figure 5-8
Joint at Fixed Bearing



5.3.5.1 Transverse Joints

Joints at fixed bearings are designed to accommodate movements of the span due to rotation of the bearing caused by loading. Typical joints at fixed bearings include silicone joints and compression seals.

Joints at expansion bearings are designed to accommodate expansion and contraction movements of the span caused by temperature changes and loading. For new construction, the two types of joints used by the Department at expansion bearings are:

- strip seals, and
- finger joints.

5.3.5.1.1 Strip Seals

Strip seals are used for new construction. In selecting strip seals, the designer must consider the relationship between total movement, minimum and maximum joint widths, and installation temperature. The application of strip seals is limited to 4 in [100 mm] of movement. Prior to specifying strip seals, the designer should confirm their availability. Strip seals less than 3 in [75 mm] should not be specified due to lack of availability and future maintenance considerations. The seal size must be adjusted to accommodate joint movement as affected by bridge skew. Because of rotation, expansion of skewed bridges may not be in a straight line. Figure 5-9 shows the details and sample design calculations for transverse strip seal joints.

5.3.5.1.2 Finger Joints

Finger joints are used where movements in excess of 4 in [100 mm] must be accommodated. A finger joint is an expansion joint with the opening spanned by meshing steel plates formed as fingers or teeth. Finger joints are open so a trough must be installed to control runoff through the joint. The gradient of the trough must be as steep as the location will permit to ensure drainage and flushing. Refer to Figure 5-10 for details.

5.3.5.2 Longitudinal Joints

Longitudinal joints in bridge decks may be required for very wide bridges, widened bridges or staged construction bridges. (A wide bridge is defined as being over 90 feet [27 m] wide or having a span to width ratio less than one, i.e. the width is greater than the span.) Longitudinal joints are always placed between beams or girders. Place them in the median or next to the median, if possible. Avoid placing longitudinal joints in the traveled way because of the hazard to motorcycles. Compression seals are not used for longitudinal joints. The designer must determine the amount of joint movement, both transverse and vertical, when designing longitudinal strip seals. Figure 5-11 shows details for longitudinal strip seals.

5.3.5.3 Construction Joints

Construction joints, either transverse or longitudinal, are permitted only at locations shown on the plans. A construction joint must be used at the break between pours, such as those required by the pouring sequence. Normally, construction joints are keyed, cold joints. See Figure 5-12.

Figure 5-9a
Transverse Strip Seal Joint Details

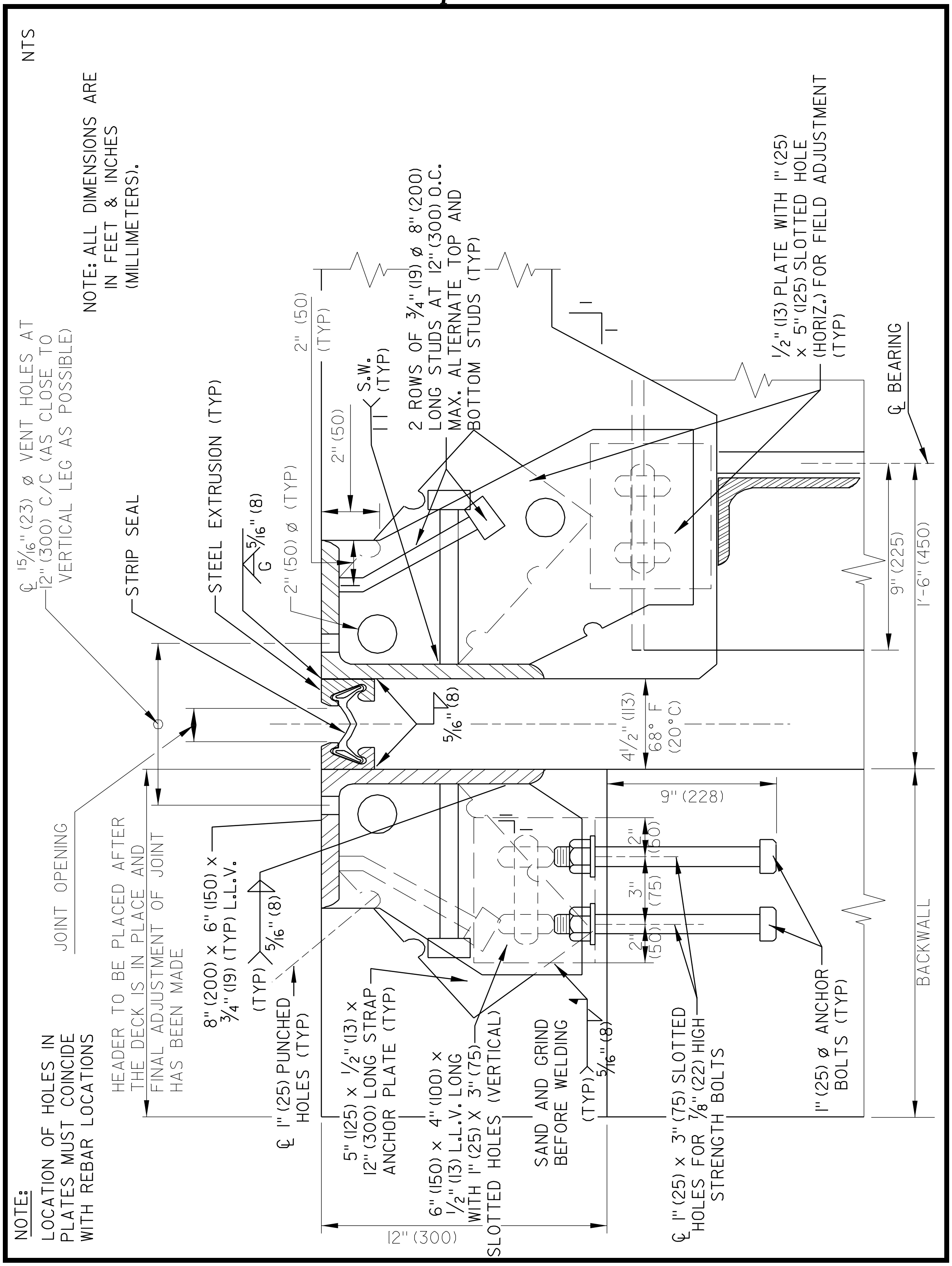


Figure 5-9b
Transverse Strip Seal Joint Details (Metric)

DESIGN VARIABLES

L = Length of structure contributing to movement
ΔL = Change in structure length due to temperature
T = Temperature
ΔT = Change in temperature (Delaware is in the moderate climate zone.)
J = Joint width
α = Coefficient of thermal expansion or contraction of a given material. Use 0.0000117 mm/mm per °C for steel and 0.0000108 mm/mm per °C for concrete.

Example Problem

Given: Steel beam bridge, 60 m, temperature range is -23°C to 43°C (for this example), so ΔT is 66°C. (See Section 5.1.5 for currently used temperature range.)
Find: Joint movement, select seal and complete joint sizing at 20 °C.

$\Delta L = \Delta T \, \alpha L$
$\Delta L = 66 \, (0.0000117 \times 60 \text{ m} \times 1000 \text{ mm/m})$
$\Delta L = 46.33$ mm Joint movement rating

Select a seal with movement rating equal to or greater than 46.33 mm.

Find the midpoint of expansion and contraction.

½ ΔT = 33°C
43°C - 33°C = 10°C

Determine the joint opening midpoint from manufacturer's literature.

1/2 Seal movement rating plus 6 mm = 44.10 mm, for this example.

Compute the joint opening at the installation temperature; assume 20°C.

$$\frac{20 - 10}{66} \times 46.33 \text{ mm} = 7 \text{ mm}$$

44.1 - 7 = 37.1 mm

Joint opening at 20°C must be 37.1 mm

Minimum joint opening:

$$\frac{43 - 10}{66} \times 46.33 = 23 \text{ mm}$$

44.1 -23 = 21.1 mm

Maximum joint opening:

$$\frac{10 - (-23)}{66} \times 46.33 = 23 \text{ mm}$$

44.1 + 23 = 67.1 mm

Figure 5-10
Finger Joint Details

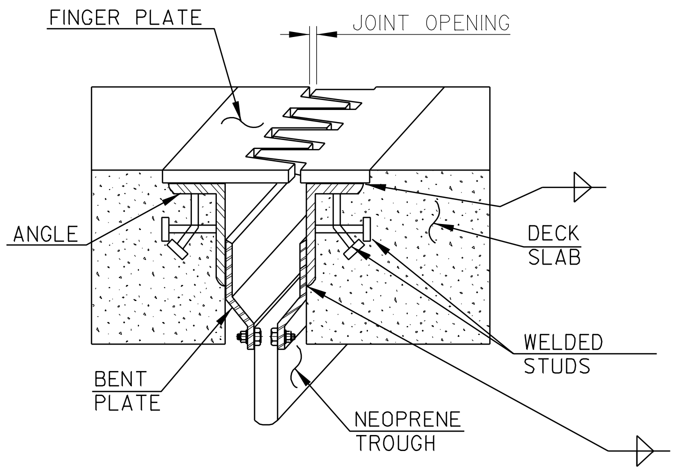


Figure 5-11
Longitudinal Strip Seal Joint Details

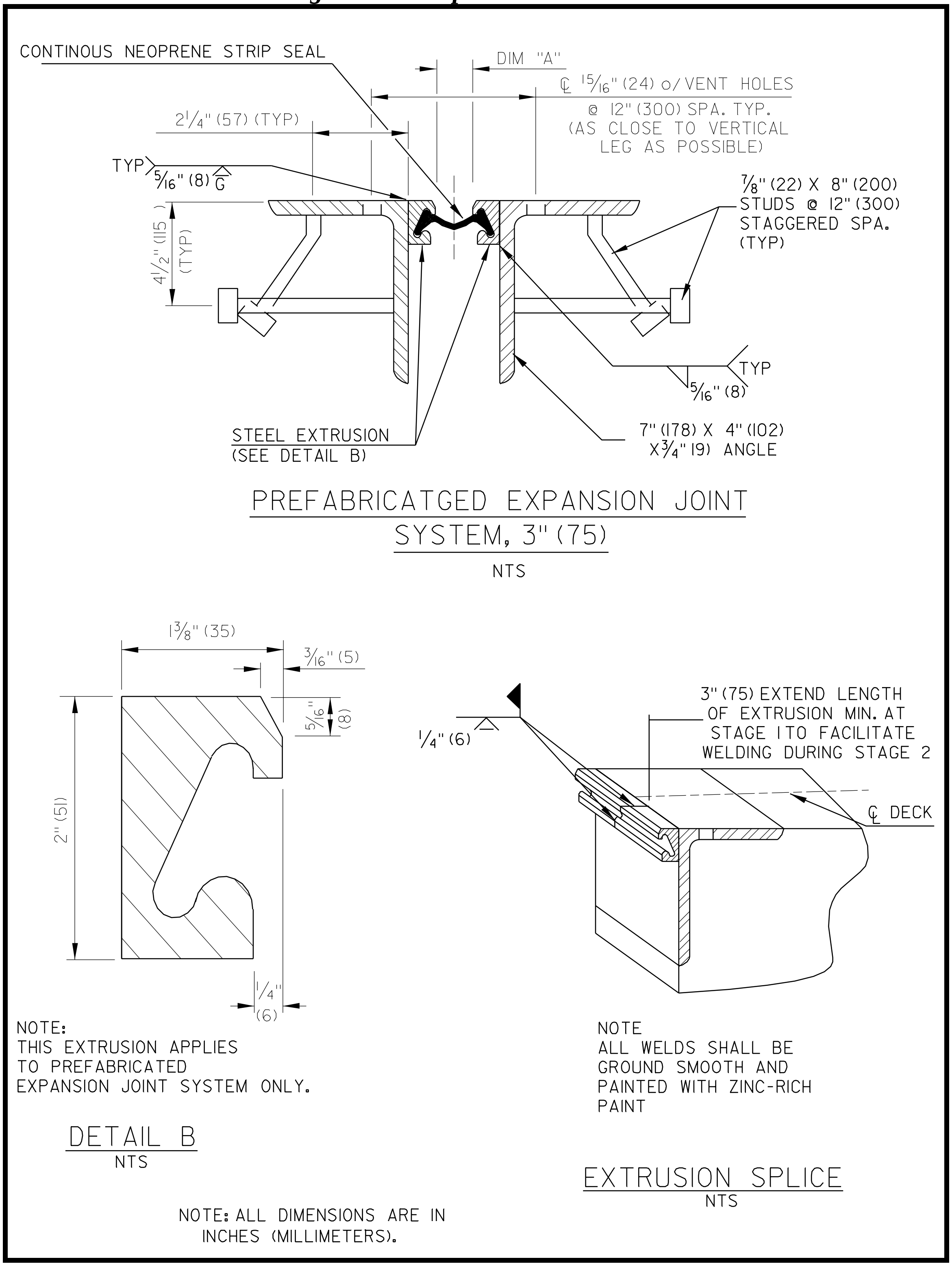
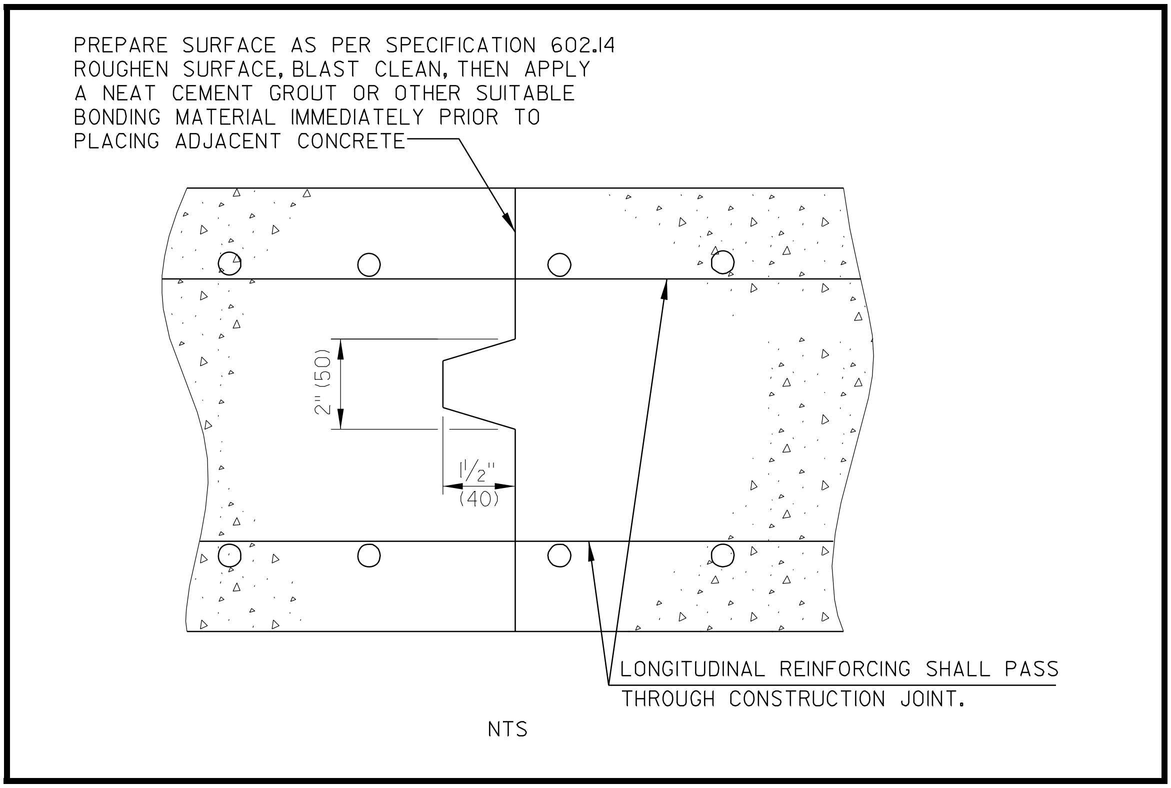


Figure 5-12
Cold Construction Joint Details



5.3.6 DECK PROTECTIVE SYSTEMS

The Department specifies concrete with low water-cement ratios, epoxy-coated reinforcing steel and extra cover for the top mat of reinforcement to ensure long-lasting decks. Additional deck protection, such as waterproofing and overlays, may be warranted on a case-by-case basis.

5.3.6.1 Overlays

Overlays are generally not constructed on new CIP decks (except for precast panel decks) because the reinforcing steel is protected through the use of additional concrete cover poured monolithically with the deck concrete. Overlays may be latex-modified concrete, silica fume concrete, or others approved by the Department. Overlays may be justified on new decks where replacement of the deck would be very costly, or where traffic would be severely impacted during deck replacement.

5.3.7 MISCELLANEOUS DECK ITEMS

5.3.7.1 Bridge Rails

5.3.7.1.1 Policies

Crash-testing. Every type of barrier used on Delaware bridges must have passed crash tests accepted by the Federal Highway Administration for the appropriate test level. All barriers must meet design criteria in the *AASHTO Specifications* and crash-testing criteria defined in *NCHRP Report 350, Recommended Procedures for Safety Performance Evaluation of Highway*

Features. Consult the current FHWA list of crash-tested details. Refer to NCHRP 350 for information regarding test levels. Traffic barriers include F-shape, vertical barriers, open ornamental barriers and aesthetically treated barriers. Pedestrian and bicycle railings must also comply with the *AASHTO Specifications.*

For high-speed highways (50 mph [80 km/h] or greater), pedestrian and bicycle traffic should have both an outside pedestrian or bicycle railing and an inside barrier railing.

Contraction joints. The maximum spacing for contraction joints in barriers and railings is 12'-0" [3.6 m]. Joints in barriers and railings at deck joints on skewed bridges may require special treatment to prevent damage to the barriers during expansion and contraction. For skews between zero and 15 degrees, the joint in the barrier should be the same as the deck joint. All other barrier joints should be perpendicular to the edge of slab. When the skew is greater than 15 degrees, the joint in the deck under the barrier must also be perpendicular to the edge of the deck.

5.2.7.1.2 Types of Bridge Rails

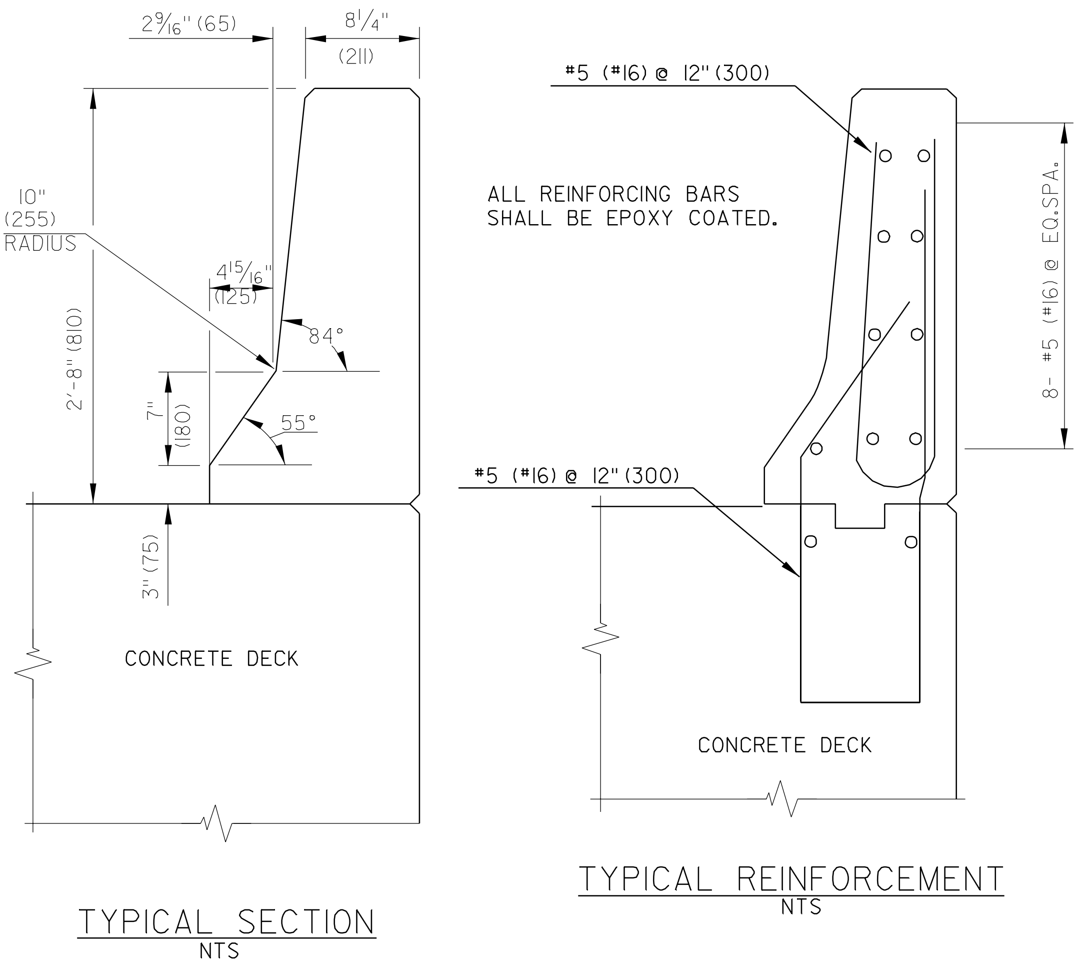
Two types of solid barriers are used on new concrete-decked bridges: F-shape and vertical face. The F-shape barriers are normally used by the Department for most bridges. Vertical barriers are used in combination with sidewalks or bicycle facilities. Vertical-face barriers may also be used where form liners or stone facings are required for aesthetics. The types of barriers and railings are:

- **F-Shape Barrier** (Figure 5-13) - The F-shape barrier is preferred for use on bridges. The standard height for Delaware's use of these barriers is 2'-8" [810 mm]. This approximates the AASHTO minimum height of 2'-5" [735 mm] plus 3 in [75 mm] for a future overlay. Higher barriers (3'-6" [1070 mm]) on bridges carrying high volumes of truck traffic may be justified with the approval of the Bridge Design Engineer. Higher median barriers may also be justified to reduce headlight glare.
- **Pedestrian Railing with Handrail** (Figure 5-14) **-** A vertical barrier with a handrail should be used on bridges with sidewalks. The height to the top of the handrail is 3'-6" [1070 mm]. The concrete section is 2'-3" [685 mm] high.
- **Bicycle Railing** - Where bicycle paths must be carried across structures, bicycle railings may be justified. The height of bicycle railings is 4'-6" [1370 mm]. The designer should contact the Bicycle/Pedestrian Coordinator to determine where bicycle paths are located.
- **Open ornamental barriers** (Figure 5-15) - Normally, open barriers are not used. They may be justified for aesthetic reasons for restoration of historic bridges or for bridges in historic areas. Types of crash-tested open face ornamental barriers include the Texas C411 and T411, and the Oklahoma Modified TR-1 Bridge Rail. The example in Figure 5-15 meets Test Level 2 requirements.
- **Barrier with Aesthetic Treatment** (Aesthetic Stone-Faced Barrier without Handrail, Figure 5-16) - Special aesthetic treatment of barriers is not normally specified. Stone or brick facing may be considered.
- **Timber railings** - See Chapter Eight.

See Figure 5-17 for handrail details.

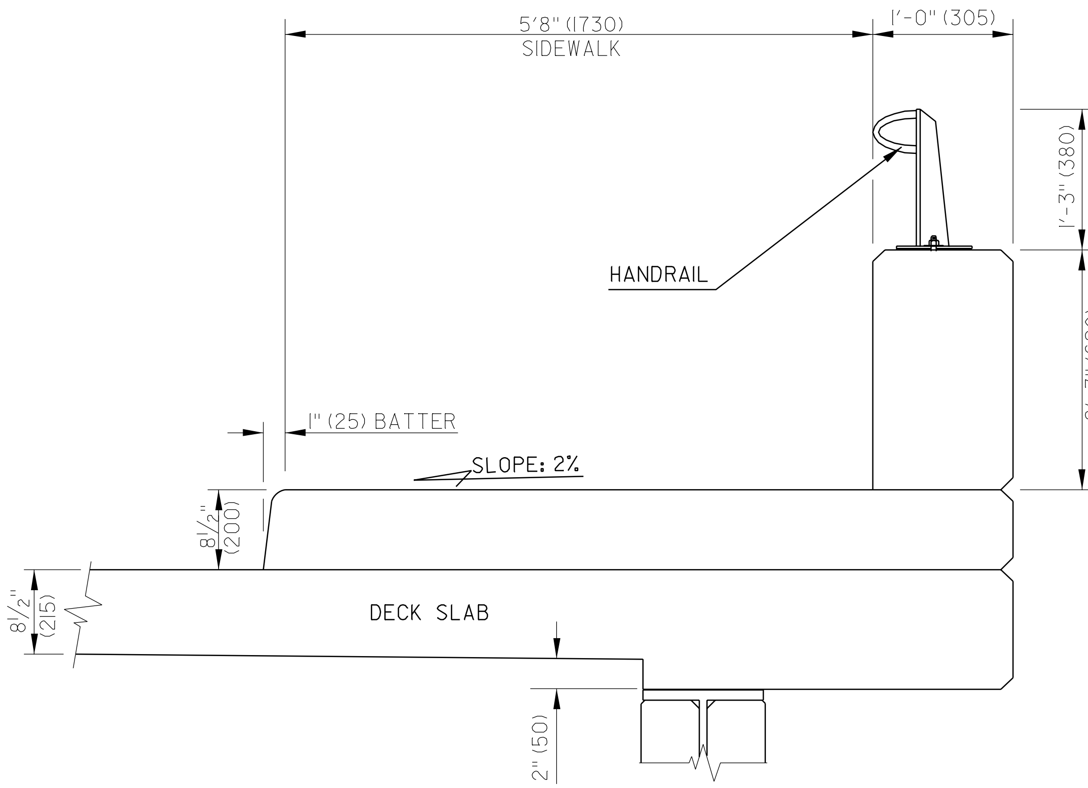
Provisions must be made to accommodate drainage at the barrier. See Section 3.3.6.

Figure 5-13
F-Shape Barrier Details



NOTE: ALL DIMENSIONS ARE IN INCHES (MILLIMETERS) UNLESS OTHERWISE NOTED.

Figure 5-14
Pedestrian Railing with Tube Handrail Details



NOTE: ALL DIMENSIONS ARE IN INCHES (MILLIMETERS).
FOR HANDRAIL DETAILS, REFER TO FIGURE 5-18B

Figure 5-15
Open Ornamental Barrier Details

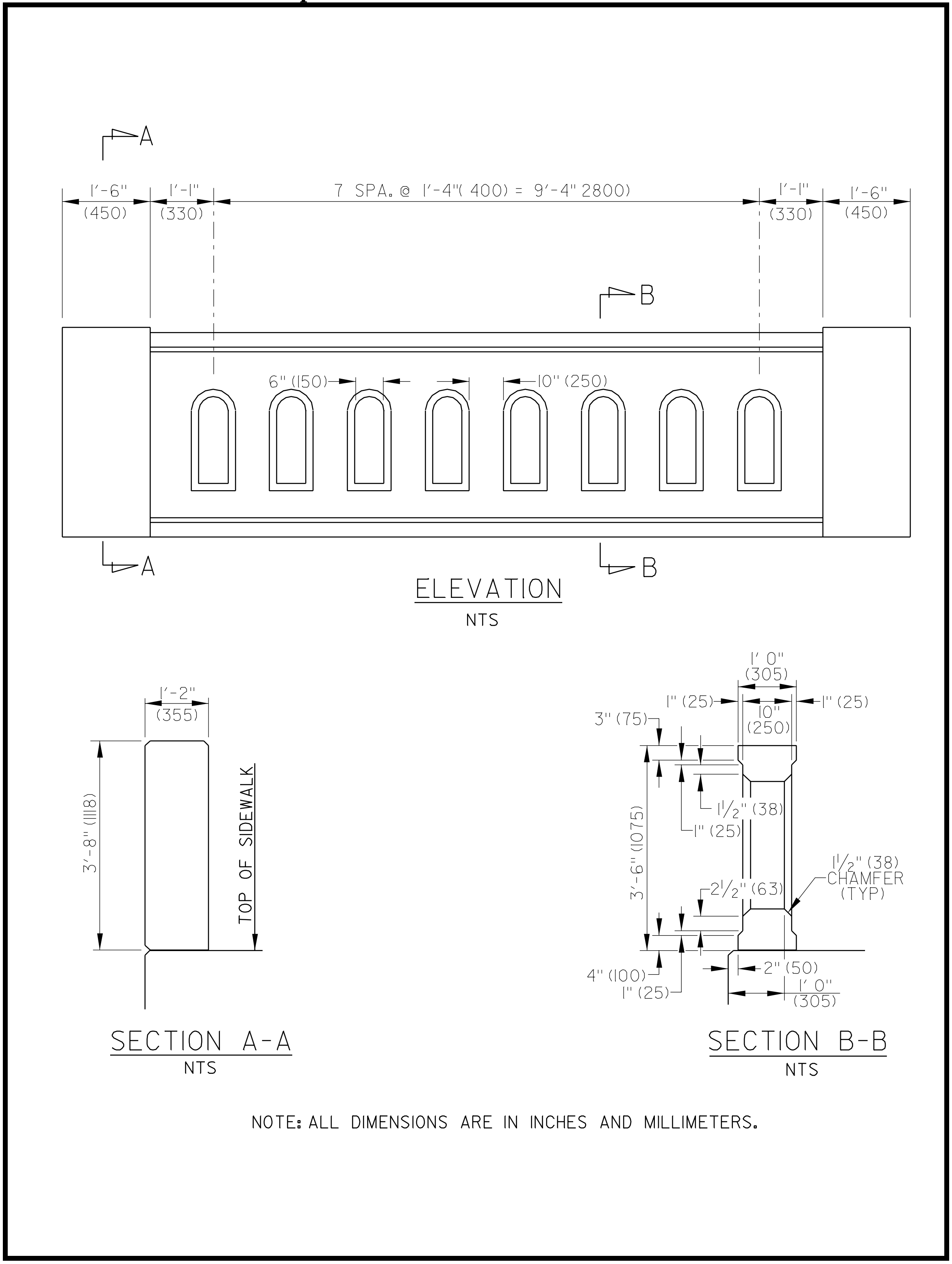


Figure 5-16
Aesthetic Stone-Faced Barrier without Handrail

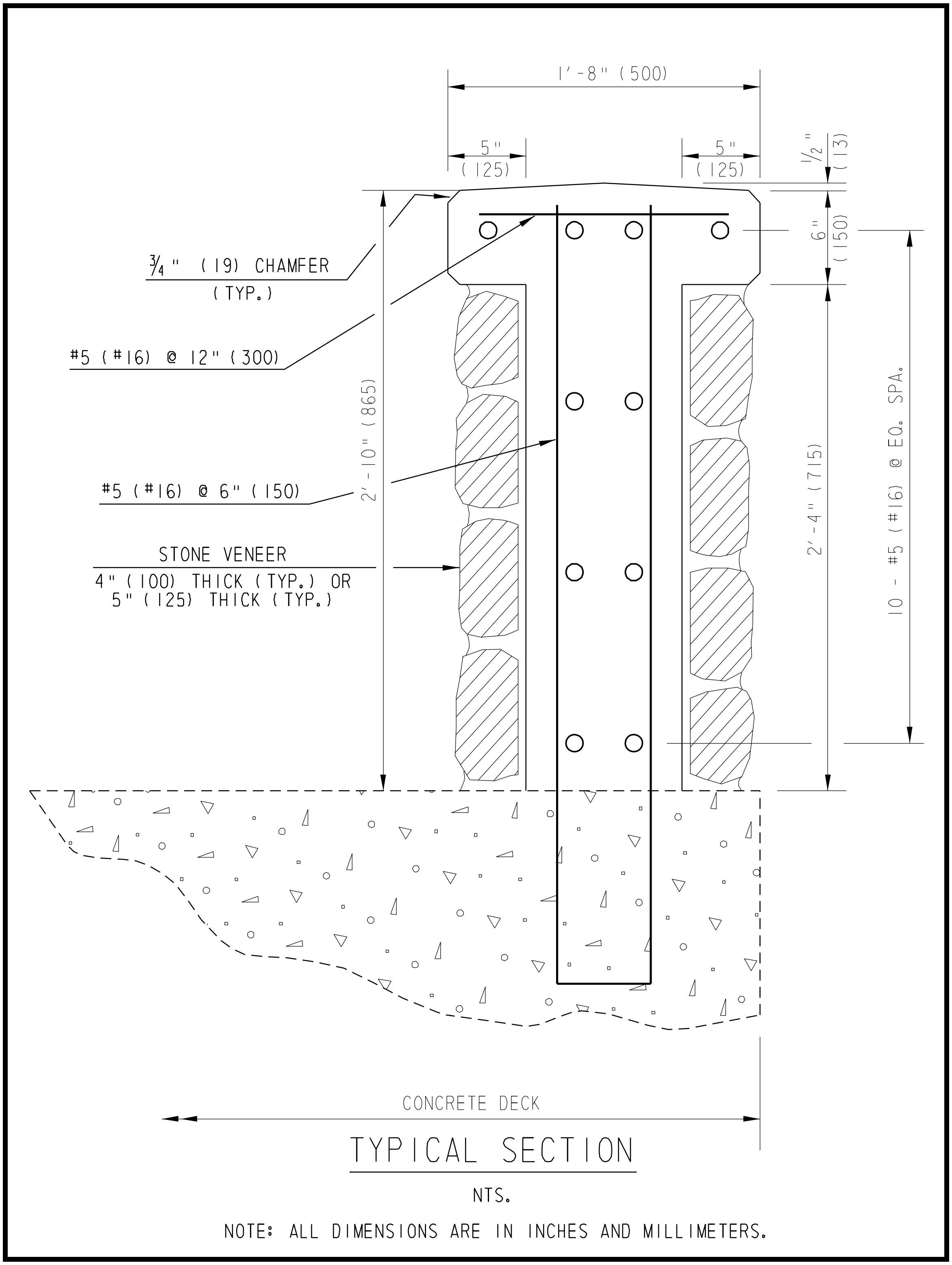


Figure 5-17a
Handrail Design Guidelines

HANDRAIL DESIGN GUIDELINES

1. Aluminum handrail assembly is Magnode Products, Inc. of Trenton, Ohio, or approved equal.

2. Posts are set perpendicular to grade.

3. Posts may be moved 0.5 in [13 mm] or less to clear expansion and deflection joints.

4. Rails are continuous from end to end of bridge with splices where rail sections cross expansion joints.

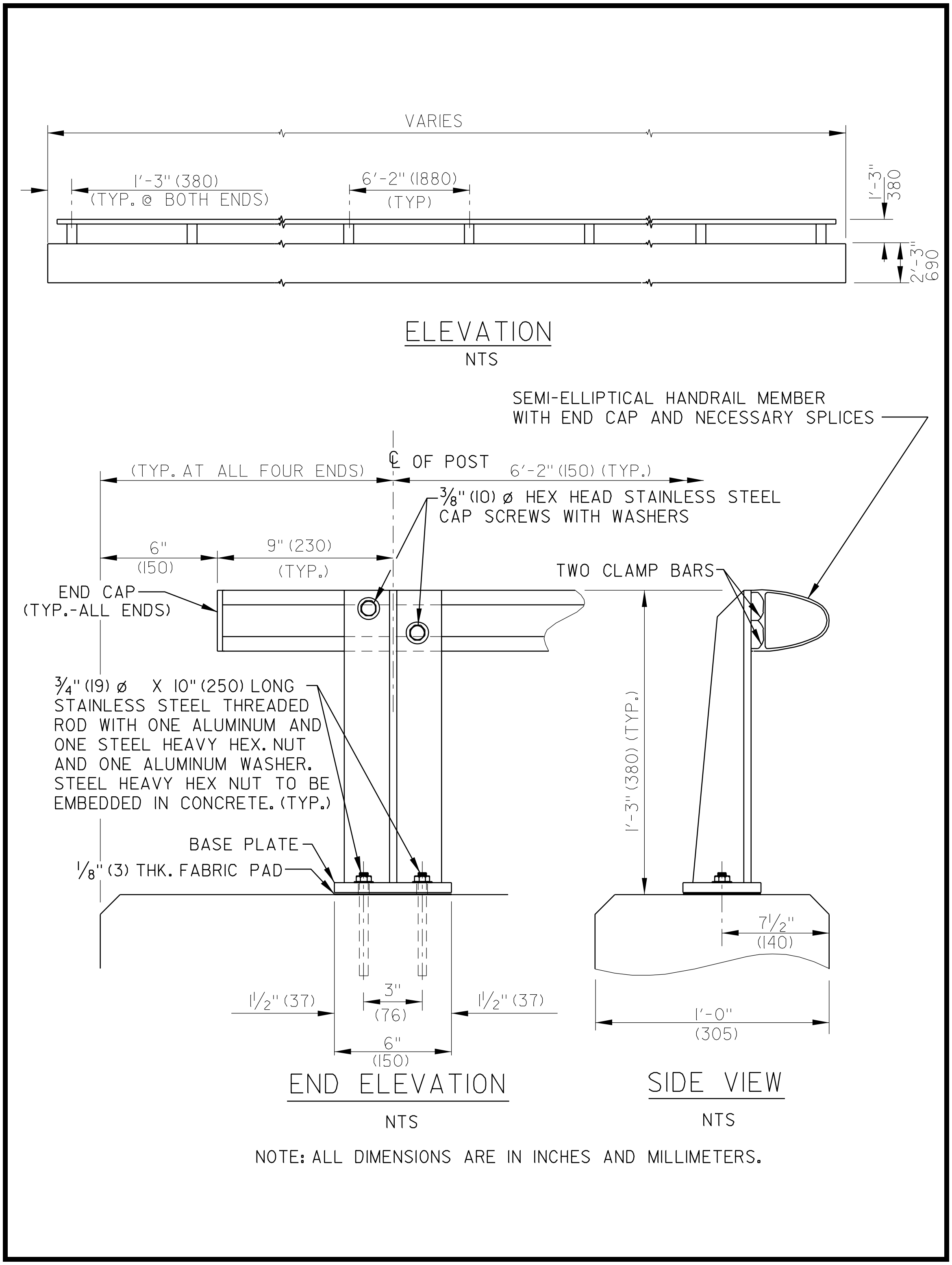
5. The centerline of any splice is located at least 2 feet [0.6 m] from the centerline of a post.

6. The bottom of post bases are thoroughly coated with an approved caulking compound or an approved zinc chromate paint.

7. Design in accordance with current AASHTO specifications.

8. Details shown in these plates are for aluminum rails. If galvanized steel rails are used, the contractor must submit details for approval by the engineer.

Figure 5-17b
Handrail Details



5.3.7.1.3 Materials

Class A portland cement concrete (f'_c = 4,500 psi [30 MPa] at 28 days) is used for concrete barriers.

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified. The minimum size of reinforcing is a #5 [16] bar.

All reinforcing steel shall be protected with fusion-bonded epoxy. Epoxy coating conforming with AASHTO M284 [M284M] shall be specified.

5.3.7.1.4 Cover

The minimum cover over reinforcing bars is 2 in [50 mm].

5.3.7.1.5 Guardrail to Barrier Connections

The post spacing for guardrail approaching a bridge is decreased to provide a greater resistance to impact. The guardrail must be solidly anchored to the bridge barrier. Refer to the DelDOT *Standard Construction Details*. The design of the connection must have been crash-tested in accordance with Section 5.2.7.1.2.

5.3.7.2 Screening and Shields

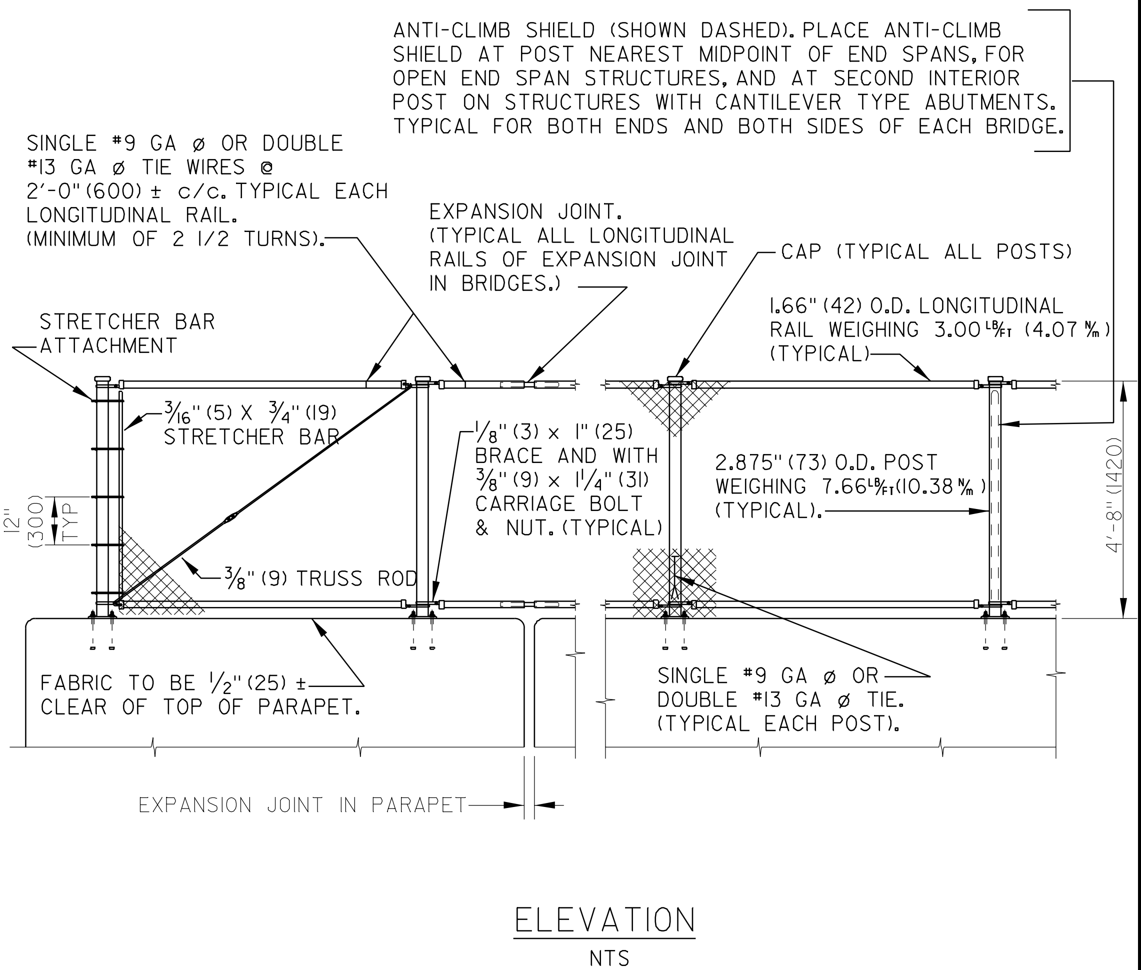
Screening is provided on selected bridges to prevent the throwing of debris onto vehicles passing beneath the bridge. Screening will be provided on a case-by-case basis. See Figure 5-18 for details. Refer to AASHTO's *A Guide for Protective Screening of Overpass Structures*.

Shields are typically required on railroad overpasses to prevent train headlights from blinding vehicle drivers. Refer to Figure 5-19.

5.3.7.3 Sidewalks

If the approach roadway has a sidewalk, the bridge sidewalk width should match the approach. Bridge sidewalks may be justified where there is no approach sidewalk. These will be evaluated on a case-by-case basis considering the need, cost, right-of-way and the possibility of future approach sidewalk construction. The standard width for sidewalks on bridges is 5'-0" [1.5 m], not including the curb. Refer to the DelDOT *Policy Implement Number O-02 - Sidewalk Policy*.

Figure 5-18a
Screening Details



NOTE: ALL DIMENSIONS ARE IN INCHES (MILLIMETERS), UNLESS OTHERWISE NOTED.

Figure 5-18b
Screening Details

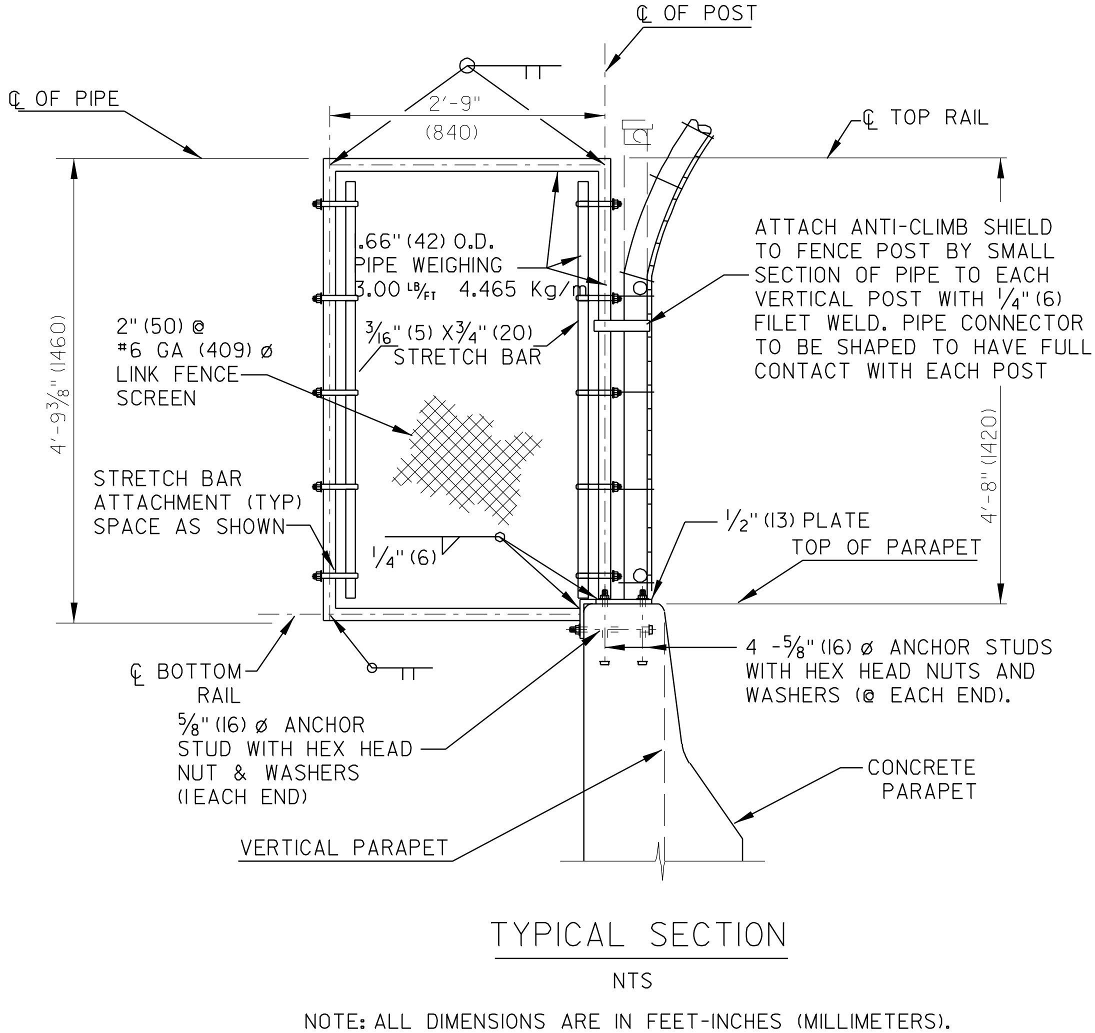
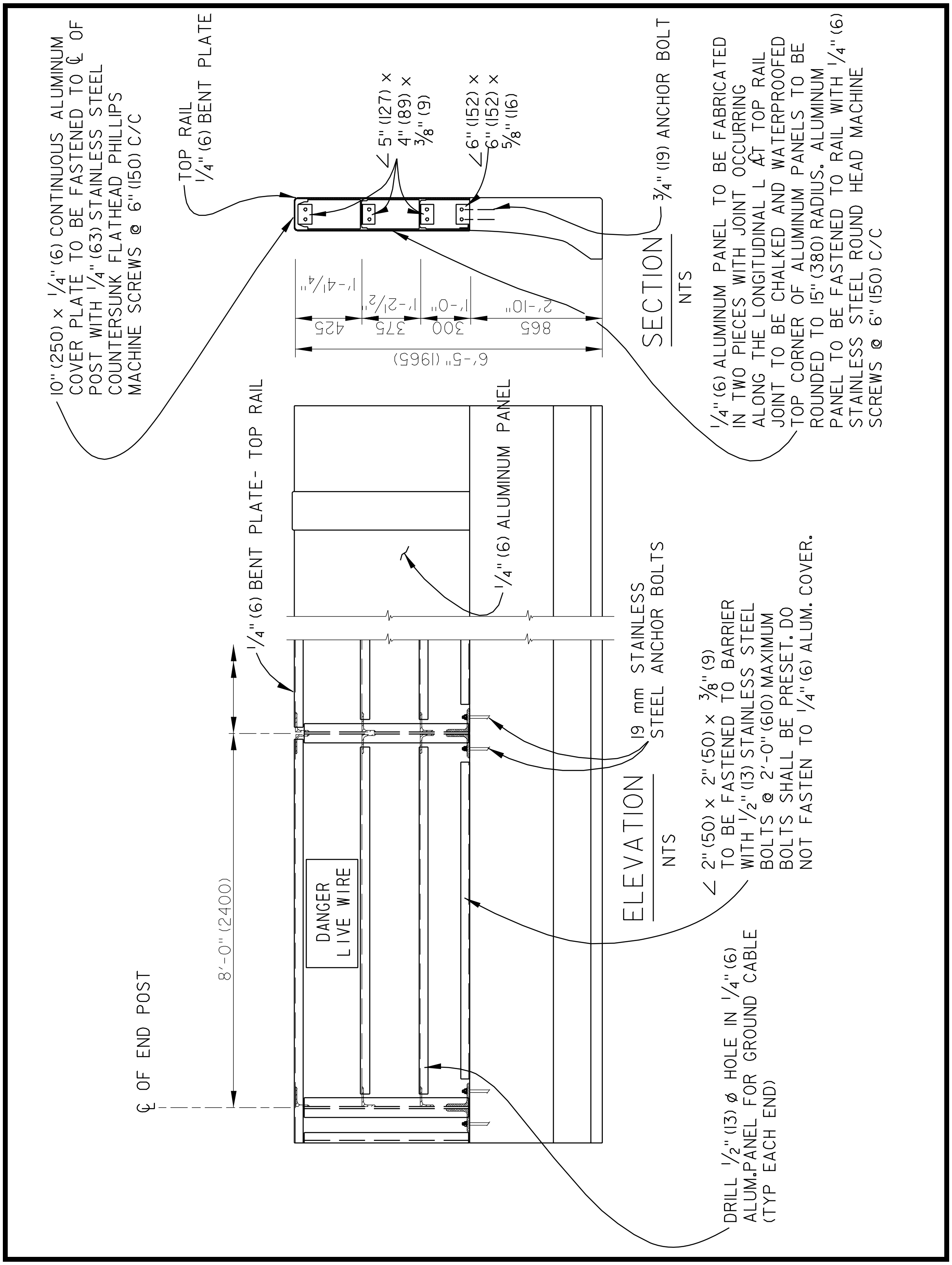


Figure 5-19
Shield Details



5.3.7.4 Curbs

Curbs are not normally used without sidewalks on Delaware bridges. A monolithic curb and sidewalk may be appropriate on low speed bridges (design speed of 45 mph [70 km/h] or lower). The curb height for the bridge must match the height of the curb on the approach roadway.

5.4 STEEL BEAMS AND GIRDERS

5.4.1 TYPES

Steel beams and girders may be straight or curved. Straight beams and girders are preferred because of simplicity of design and lower fabrication costs. The following types of steel beams and girders are used in Delaware:

- rolled I-beams;
- welded-plate girders;
- haunched girders; and
- steel box girders.

Rolled I-beams are used for spans up to 90 feet [27 m]. Their advantages are:

- economical use of material for shorter spans;
- the simplicity of construction results in savings; and
- straightforward design.

Welded-plate girders are used for spans greater than 90 feet [27 m]. Their advantages are:

- simpler to design than haunched or box girders;
- the simplicity of construction results in savings over haunched or box girders; and
- fabrication is easier and can be more automated.

Hybrid girders utilizing high-performance steel could be used for longer spans.

Haunched girders are used for spans greater than 90 feet [27 m], and where:

- vertical clearance cannot be attained with welded-plate girders; or
- aesthetics are considered.

Haunched girders are also used for spans greater than 130 feet [40 m], where:

- a variable section depth is structurally more efficient; and
- longer spans permit fabrication and materials cost savings.

Portions of haunched girders, such as cross frames and wind bracing, require special fabrication.

Steel box girders are three-sided steel boxes with a composite reinforced concrete deck. The advantages of box girders are:

- they have clean lines and pleasing appearance;
- shorter total section depth is possible than with multi-beam plate girders; and
- optimization of materials usage.

The disadvantages are that:

- the beams are difficult to lift into place due to their size and weight;
- redecking is more complex because of the need to maintain traffic and stage construction during deck removal and replacement (with composite design, the deck serves as an element of the compression flange);
- inspection and maintenance of the interior of box girders is more difficult; and

- complex geometric control is required for fabrication and construction.

The use of steel box girders in Delaware is discouraged because of the difficulty in construction and maintenance.

5.4.2 MATERIALS

The material for all main load-carrying members of steel bridges subject to tensile stresses shall meet AASHTO requirements for notch toughness. Refer to Section 6.6.2, Fracture, in the *AASHTO Specifications*.

Normally, AASHTO M270, Grade 50 [M270M, Grade 345], structural steel is used. Painting is required.

AASHTO M270, Grade 50W [M270M, Grade 345W], structural steel weathers to preclude the need for painting. Weathering steel may be considered for structures over high traffic volume roadways or railroads where access for painting or repainting is limited or dangerous. The use of weathering steel will be evaluated on a case-by-case basis and is subject to approval of the Bridge Design Engineer. Refer to the FHWA report *Forum on Weathering Steel for Highway Structures: Summary Report*. Weathering steel should not be used in corrosive environments where there is high humidity or high concentrations of chloride. The site should be tested for chloride before specifying weathering steel. It is required to paint the ends of weathering steel beams near bearings and under joints. Refer to Section 5.3.5 for painting of structural steel, including the ends of weathering steel beams.

5.4.3 DESIGN

Design references are given in Section 6, Steel Structures, in the *AASHTO Specifications*.

5.4.3.1 Redundancy Requirements

Whenever possible, the superstructure should be a redundant design. A redundant structure has multiple load paths available to share the loads should a single member fail.

Non-redundant structures are fracture-critical. A fracture-critical structure is a structure where the failure of a single tension member or tension component of a member will cause failure of the span. Fracture-critical designs should be avoided if at all possible. If a design contains fracture-critical members, these members must be specifically identified on the plans.

It is Department policy to avoid pin and hanger connection designs because of the difficulty of inspecting and maintaining them.

5.4.3.2 Beam Spacing

To support the redundancy requirements, the Department has adopted the following spacings for steel beams:

- Minimum - 8'-0" [2.4 m]
- Desirable - 9'-0" [2.7 m]
- Maximum - 10'-0" [3.0 m]

Where vertical clearance is not a problem, a wider maximum spacing (up to 12'-6" [3.8 m]) may be justified with the approval of the Bridge Design Engineer on a case-by-case basis.

5.4.3.3 Fatigue Design Criteria

See Section 6.6, Fatigue and Fracture Considerations, in the *AASHTO Specifications*. The designer must be aware of the details of built-up members and connections fabricated by welding. Certain details can significantly impact the allowable range of stress by creating a connection or splice that has a very low fatigue threshold. When a member fails due to fatigue, a crack forms which can grow without increased stress and potentially cause failure of the member. Also see AASHTO's *Guide Specifications for Fatigue Design of Steel Bridges* and FHWA's *Economical and Fatigue Resistant Steel Bridge Details*.

The designer should be aware of allowable fatigue stress range criteria as shown in AASHTO as well as the weld detail categories and the kinds of stresses (tension, shear, or reverse). See Section 6.6.2, Fracture, the *AASHTO Specifications*. Delaware is in Temperature Zone Designation 2 for Charpy V-notch impact requirements.

5.4.3.4 Beam Design

The Department follows AASHTO design procedures for beam design. One exception is cover plate design. The Department does not permit the use of cover plates.

Use a minimum flange plate thickness of 0.75 in [19 mm] and width of 12 in [305 mm] to reduce warping during fabrication, improve transportation stability, and reduce erection problems.

5.4.3.5 Welding and Welding Procedures

The Department follows the *ANSI/AASHTO/AWS D1.5 Bridge Welding Code* for welding design. Electroslag welding is one exception to the Code; its use is not permitted for Delaware bridges.

Fillet welds are preferred over other types of welds because they are easier to make with automated welding equipment and grinding may be reduced in preparing the steel plates for welding. Welds should be designed and details prepared which minimize the need for backing bars.

5.4.3.6 Shear Connectors

Stud-type shear connectors are used for both positive and negative moment areas. Studs with a 0.875 in [22 mm] diameter are used. In negative moment areas, the maximum stud spacing is 2'-0" [600 mm]. See Figure 5-20.

5.4.3.7 Stiffeners, Diaphragms, and Bracing

Two types of stiffeners are used: transverse and longitudinal.

Transverse stiffeners may be either intermediate or bearing. For girders with webs 4'-6" [1.37 m] or smaller, it is preferable not to use intermediate stiffeners. For girders with webs larger than 4'-6" [1.37 m], the web thickness may be increased to limit the transverse stiffeners to only one or two locations per span beyond those provided for diaphragm or cross frame connections. Transverse stiffeners must be a minimum of 0.375 in [9.5 mm] thick. Stiffeners shall be welded to the web with a minimum 0.1875 in [4.8 mm] continuous fillet weld. Intermediate stiffeners will be welded to the compression flange and tight fit to the tension flange. Bearing stiffeners shall be welded to both the top and bottom flanges. Transverse stiffeners used as connection plates for diaphragms will be welded or bolted to both flanges, and the flange stress shall be investigated for fatigue. See Figures 5-21 to 5-27 for details. Transverse stiffeners will be clipped as shown in the figure.

Figure 5-20
Shear Developer – Steel Beams

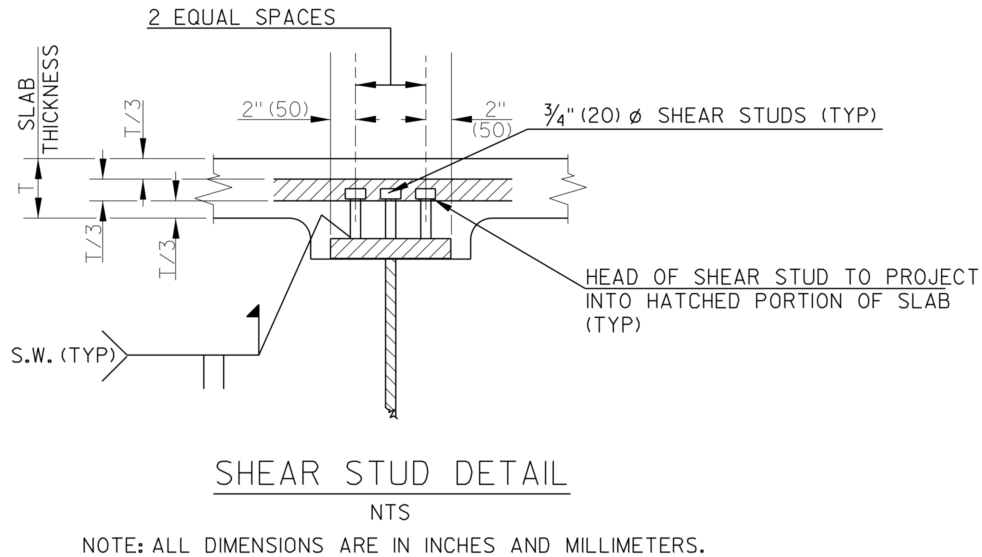
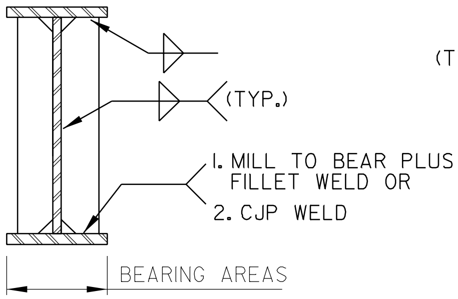
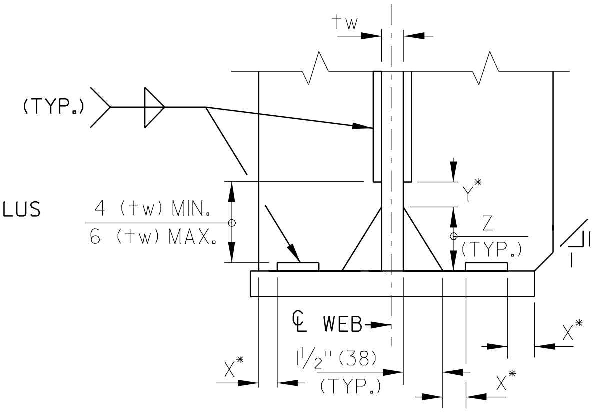


Figure 5-21a
Intermediate and Bearing Stiffener Details – Steel Beams



BEARING
STIFFENER



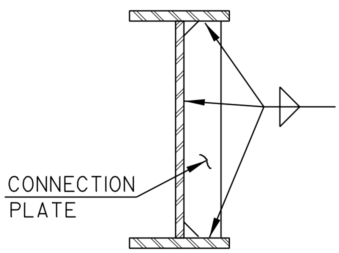
• SHOW STIFFENER PLATE & FILLET WELD SIZES ON THE PLAN

X = ¼" (6) ± ⅛" (3)
Y = ½" (13) ± ¼" (6)

Z = { 2½" (64) FOR ½" (13) WEB
3" (75) FOR 9/16" (14) WEB
4" (100) FOR ¾" (19) WEB }

*0 mm FOR GROOVE WELD.

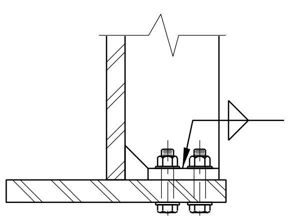
DETAIL @ END OF STIFFENER
OR CONNECTION PLATE



CONNECTION
PLATE

NOTES :

A. STIFFENER SIZE MUST BE SHOWN ON PLANS.
B. FILLET WELD SIZE SHALL BE SHOWN ON PLANS.
C. IF A BEARING STIFFENER IS USED AS A CONNECTION PLATE FOR CROSS FRAMES. FILLET WELDS ARE REQUIRED.
D. WHEN LONGITUDINAL STIFFENERS ARE REQUIRED, PLACE ALL TRANSVERSE STIFFENERS ON ONE SIDE OF WEB AND PLACE THE LONGITUDINAL STIFFENER ON OPPOSITE SIDE.



ALTERNATE DETAIL
@ TENSION FLANGE
WHERE STRESS RANGE
EXCEEDS CATEGORY C

Figure 5-21b
Intermediate and Bearing Stiffener Details – Steel Beams

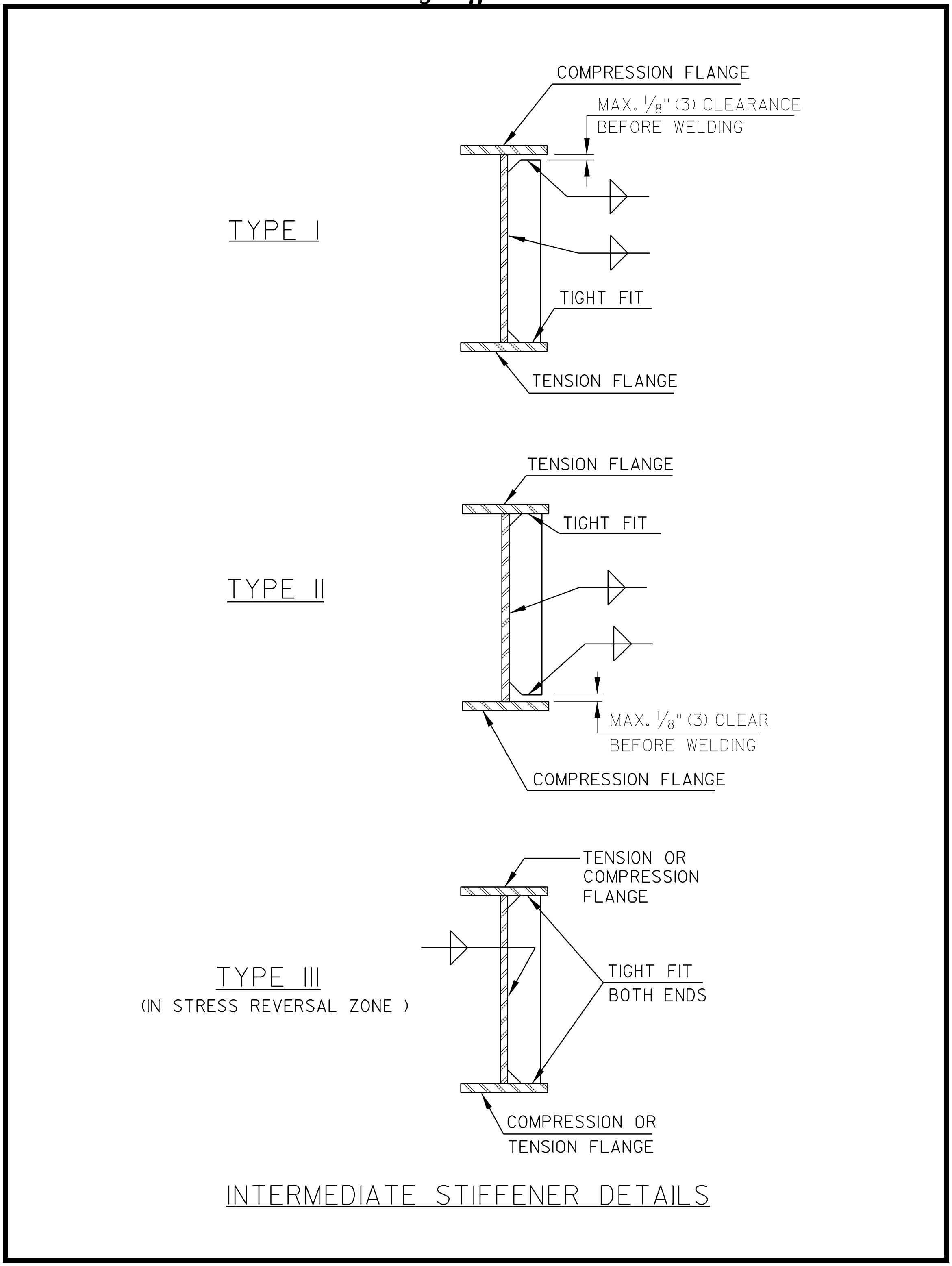


Figure 5-22
Cross Frame Details – Steel Beam Example

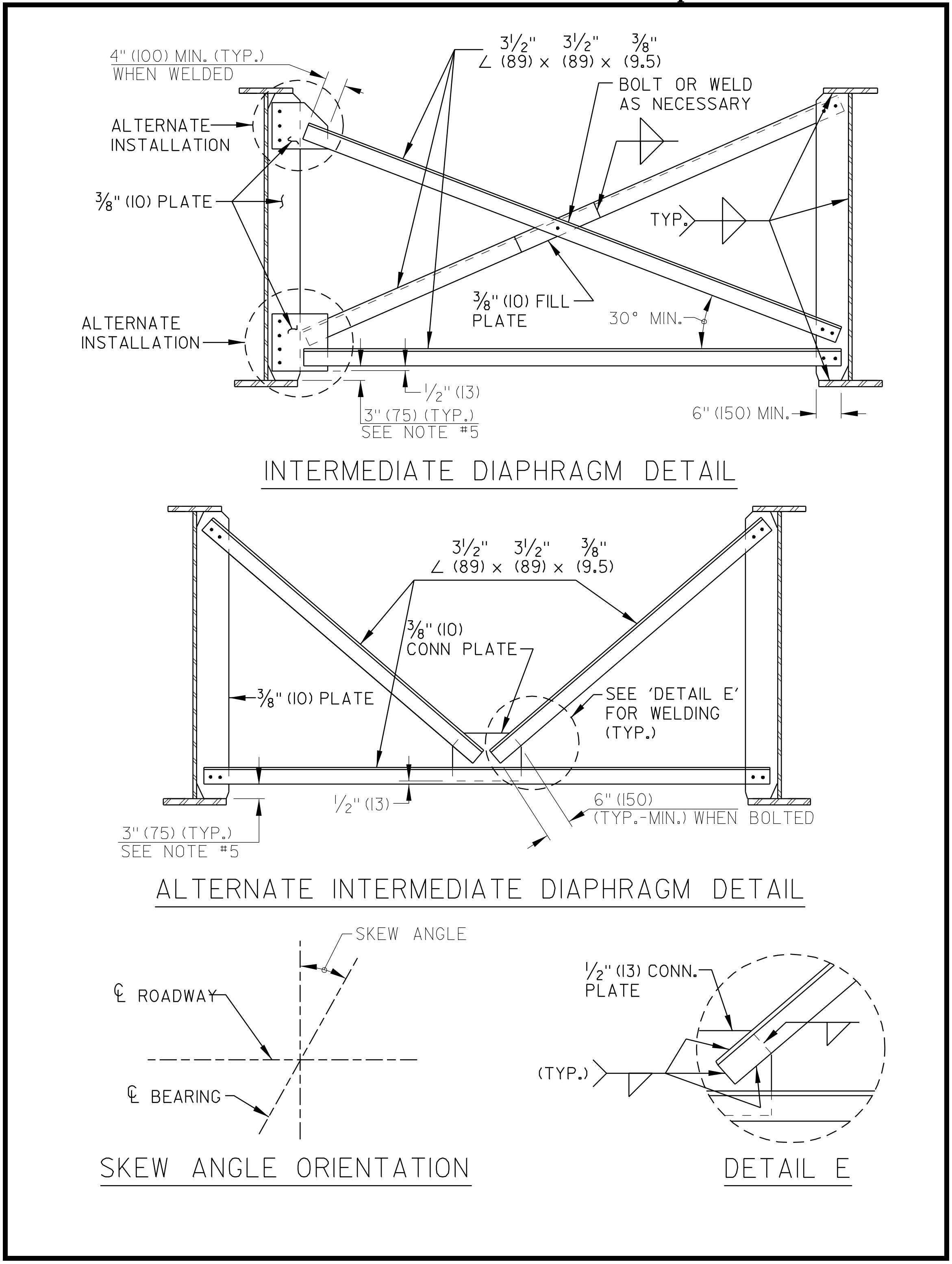


Figure 5-23a
End Cross Frame Details – Steel Beam Example

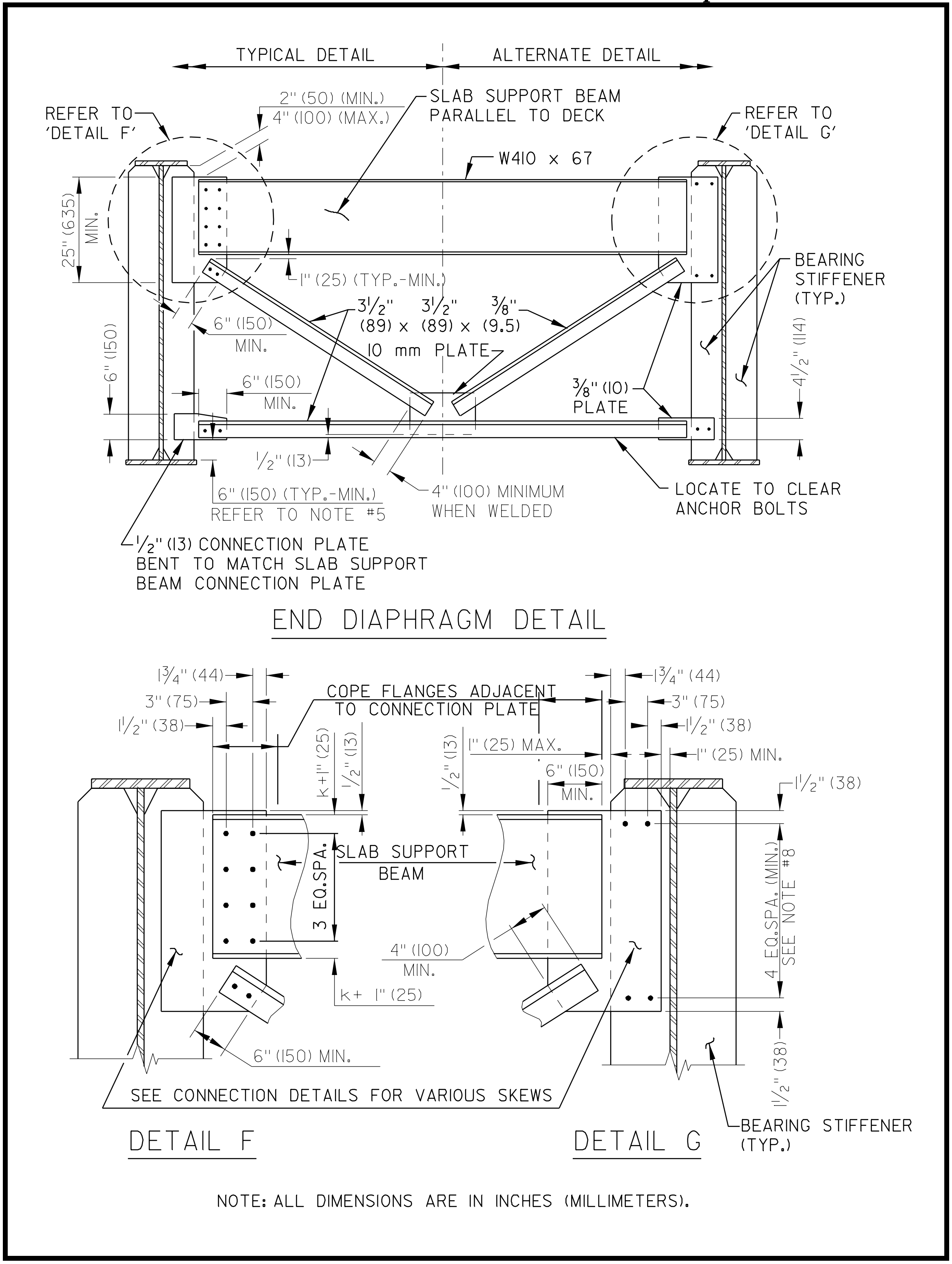


Figure 5-23b
End Cross Frame Details – Steel Beam Example

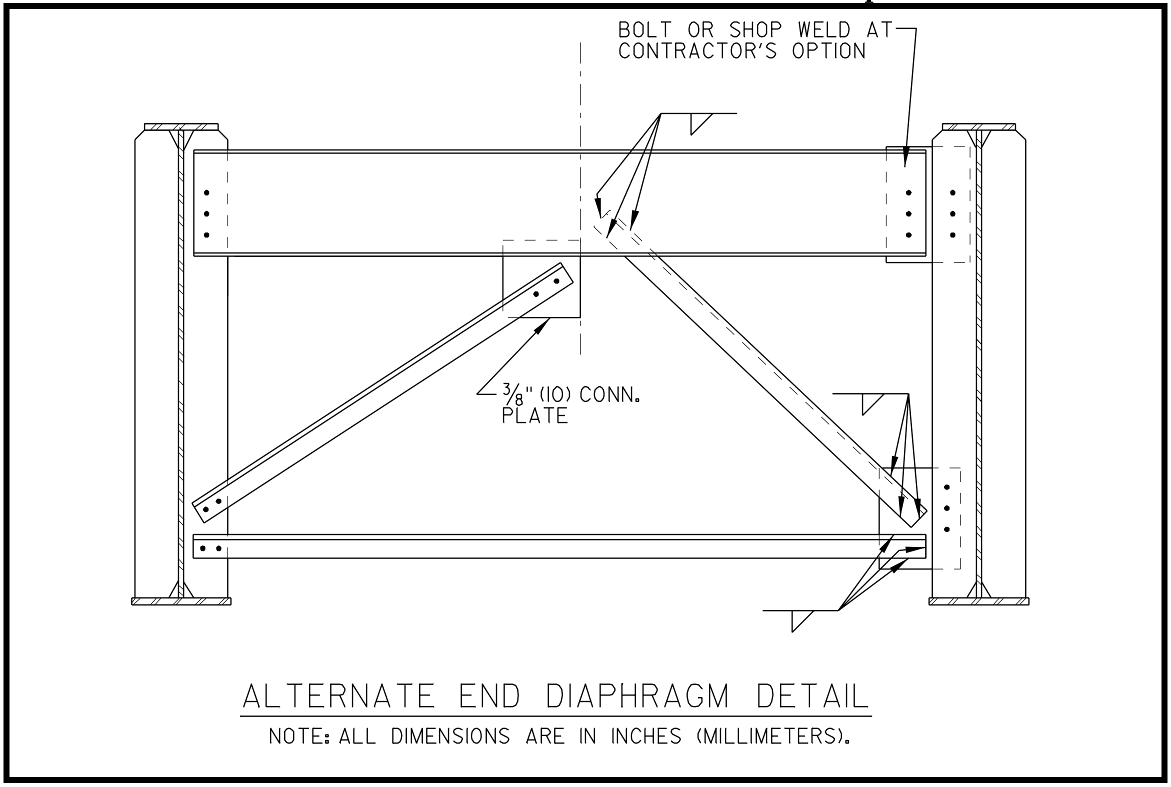


Figure 5-24a
Connection Plate Details – Steel Beam Example

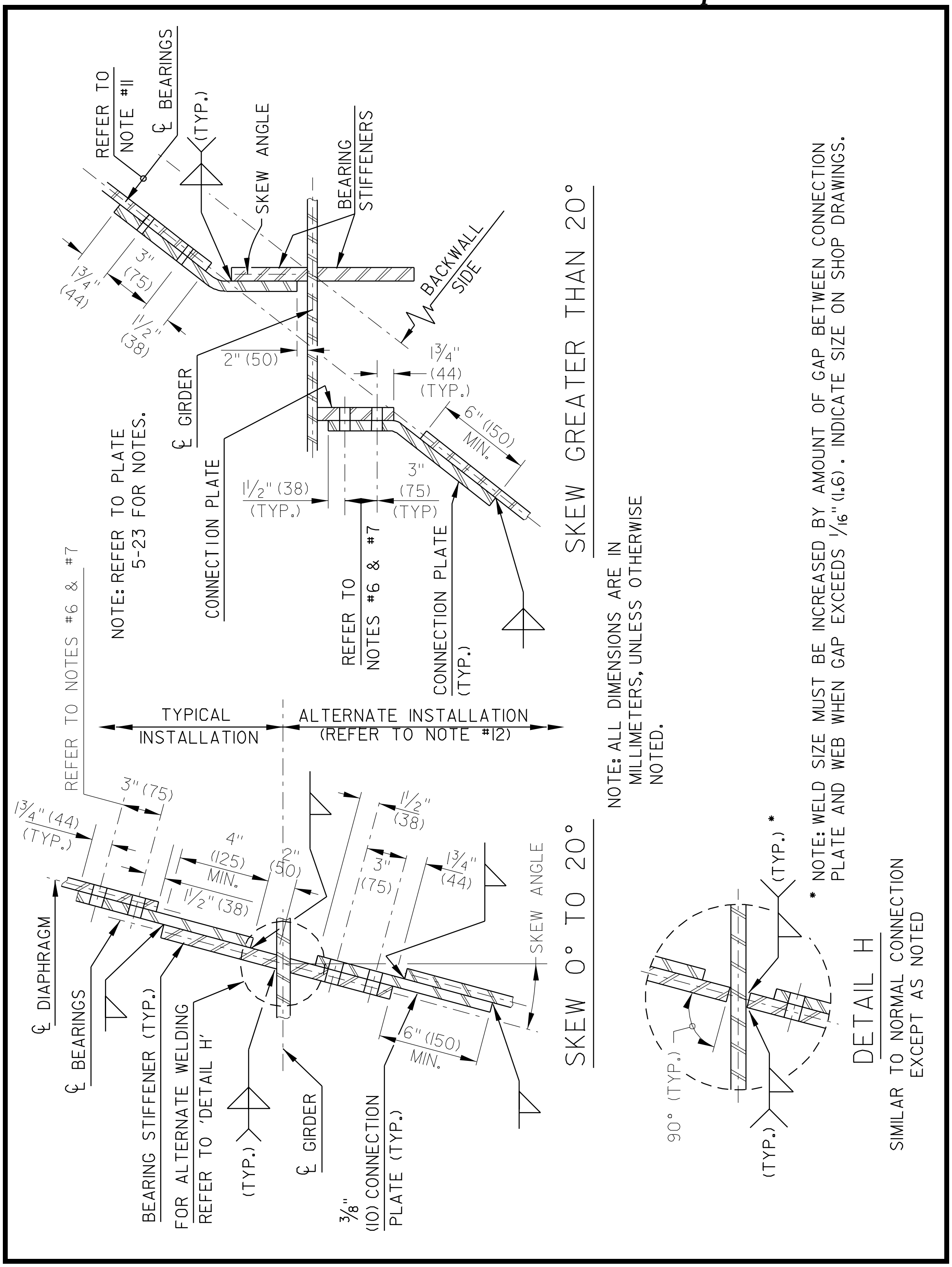


Figure 5-24b
Connection Plate Details – Steel Beam Example

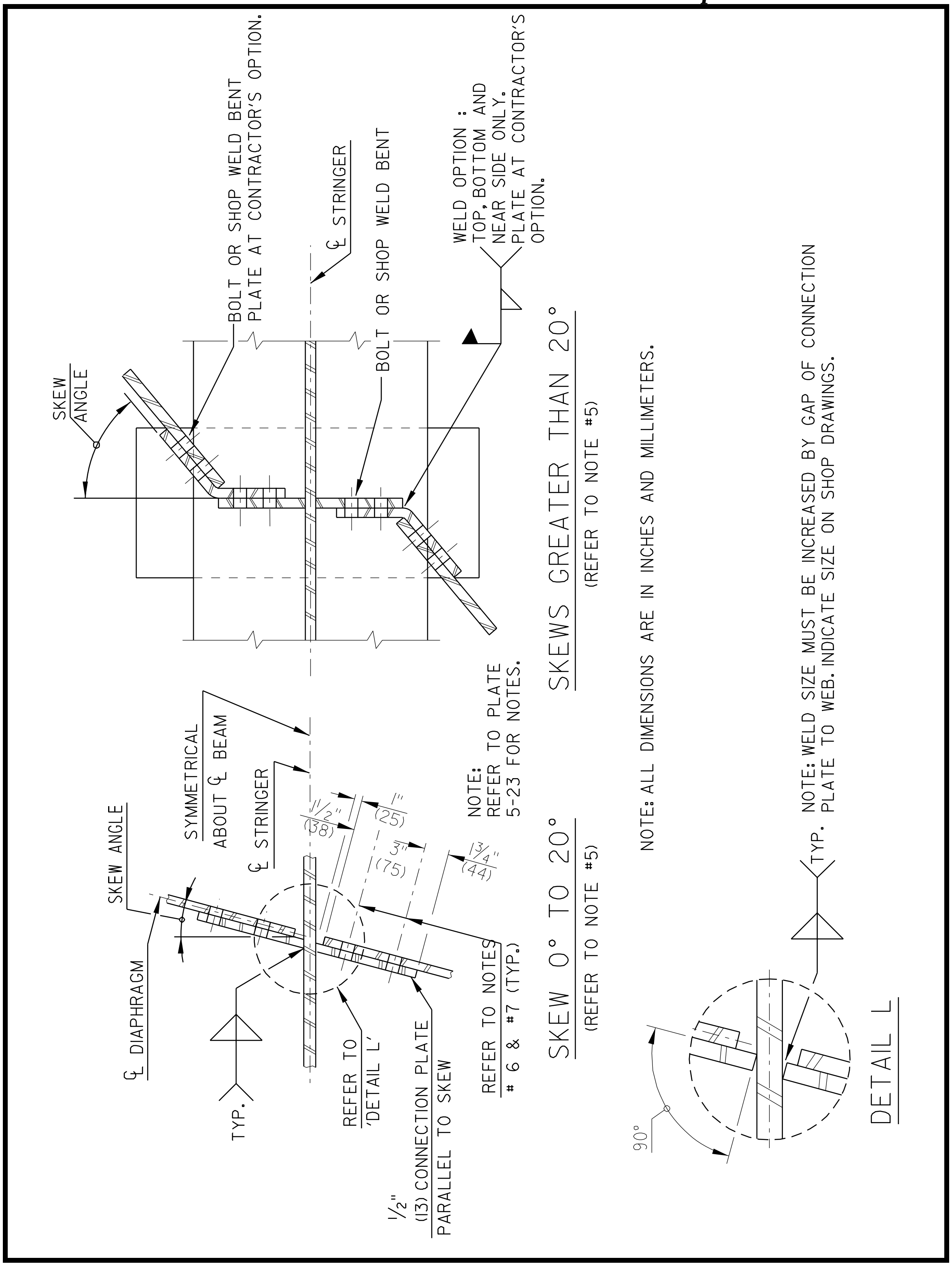


Figure 5-25
Lateral Bracing Details – Steel Beam Example

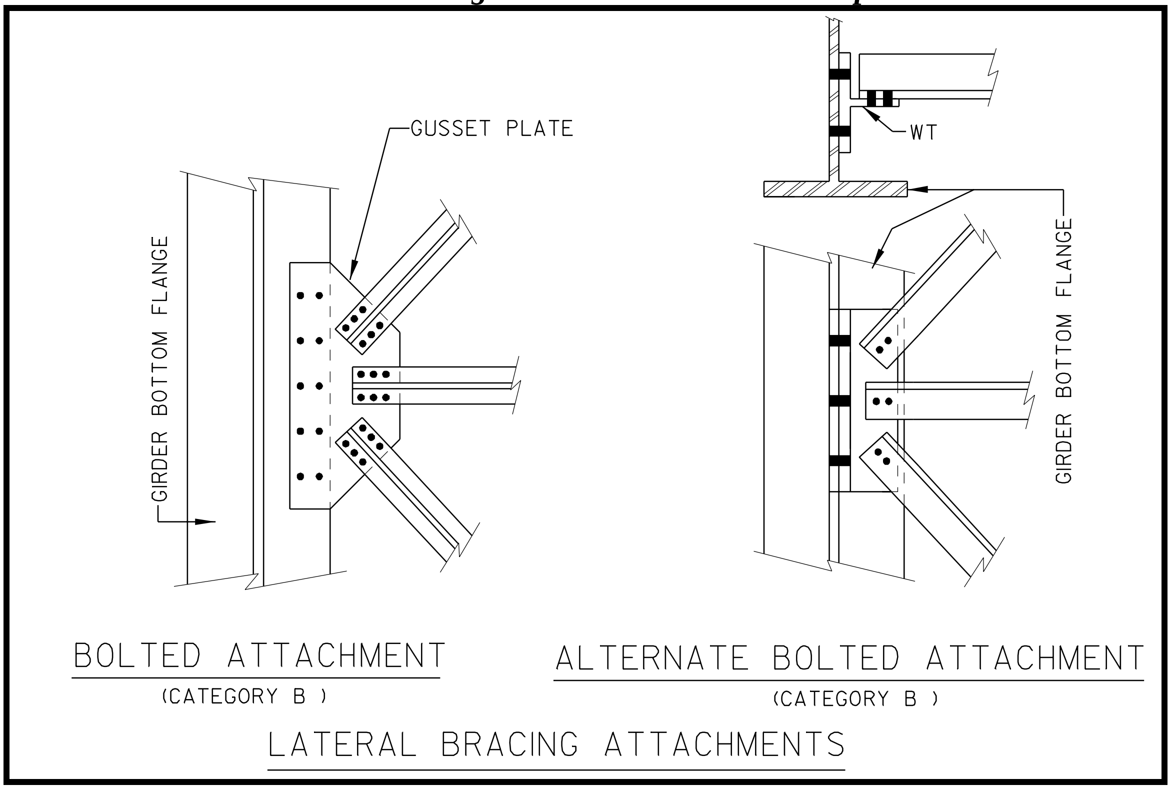


Figure 5-26a
Rolled Beam Intermediate Diaphragm Detail Example

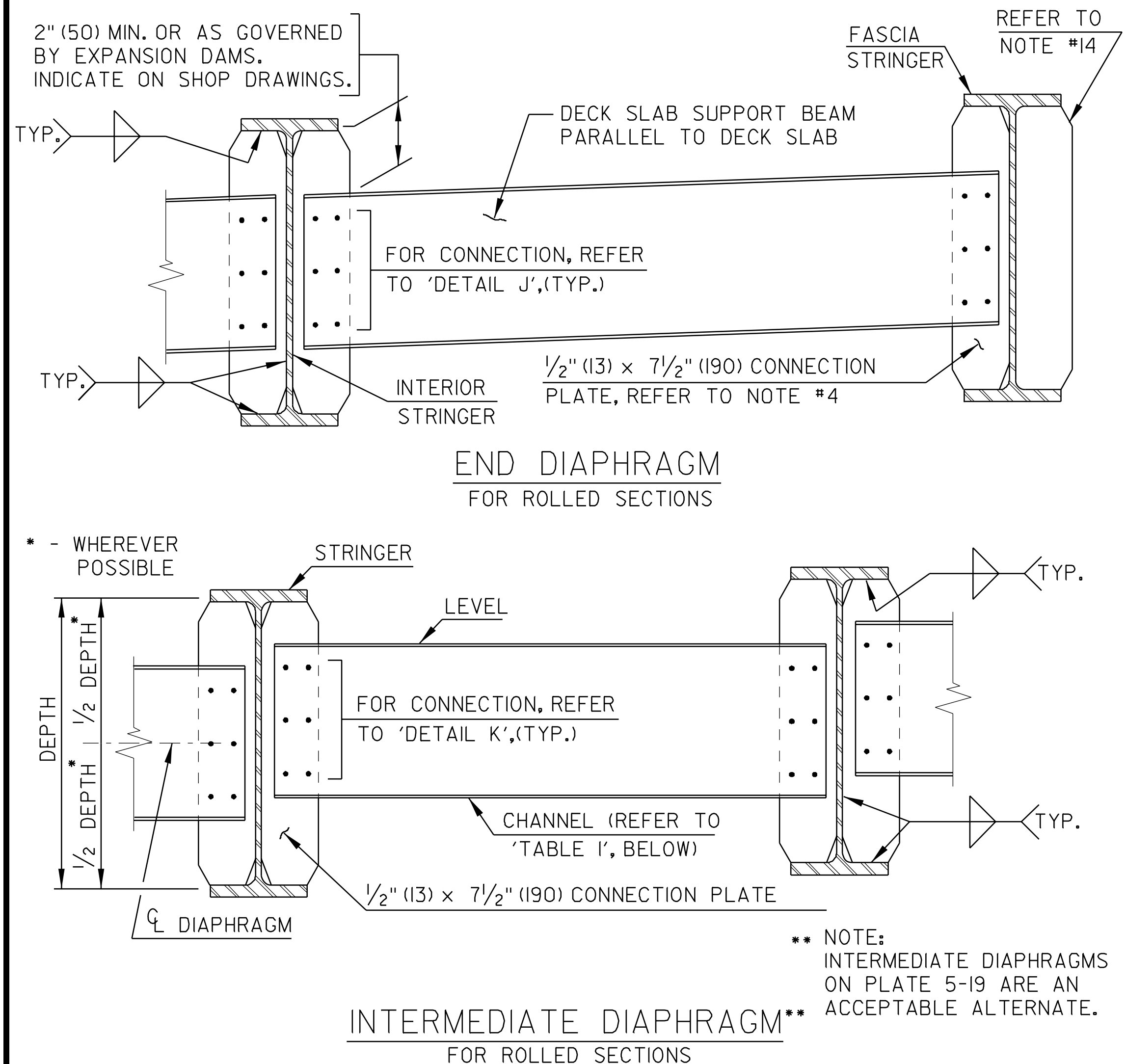


TABLE I

STRINGER SIZE	DIAPHRAGM SIZE	NO. OF BOLTS
≥ 27" (690) DEPTH	C15" X 33.9 (C 380 x 50)	8
UP TO 24" (610) DEPTH	C12" X 25 (C 310 x 37)	6

NOTE: ALL DIMENSIONS ARE IN INCHES (MILLIMETERS).

Figure 5-26b
Rolled Beam Intermediate Diaphragm Detail Example

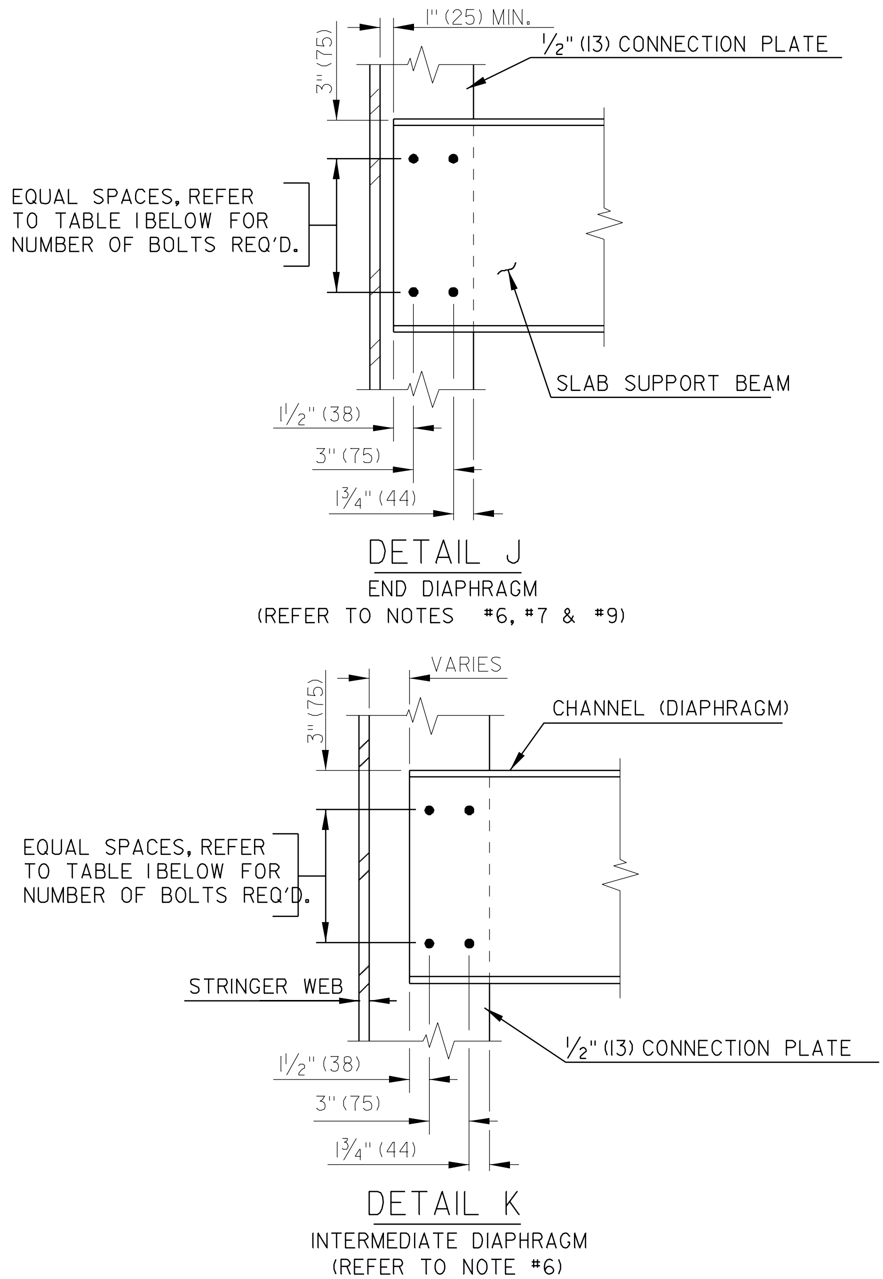


TABLE I

STRINGER SIZE	DIAPHRAGM SIZE	NO. OF BOLTS
≥ 27" (690) DEPTH	C15" X 33.9 (C 380 × 50)	8
UP TO 24" (610) DEPTH	C12" X 25 (C 310 × 37)	6

NOTE: ALL DIMENSIONS ARE IN INCHES (MILLIMETERS).

Longitudinal stiffeners are used to improve the bending resistance of welded-plate girders. Because of the increased fabrication complexity and greater likelihood of occurrence of welds or weld-intersection flaws, longitudinal stiffeners may be used only with the approval of the Bridge Design Engineer. The longitudinal stiffeners should always be placed on the opposite side of the web from the transverse intermediate stiffeners to minimize the number of intersections between longitudinal and transverse stiffeners. Transverse intermediate stiffeners used for diaphragm connection plates must be placed on both sides of interior beams. Longitudinal stiffeners may not be continuous and may be cut at their intersections with transverse stiffeners. Close attention to the details at the intersection of longitudinal and transverse stiffeners is needed. Avoid intersecting welds, if possible, by stopping the welds short of the intersection. Where intersecting welds cannot be avoided, nondestructive testing (NDT) must be specified to detect weld flaws which may cause cracking. Refer to Figure 5-27.

Diaphragms, cross frames, and lateral bracing are used to stiffen and connect beams so they work as a unit. Diaphragms are used to connect rolled beams with 3'-0" [0.915 m] height or less. Channel diaphragms for rolled beams shall be at least half beam depth. Cross frames are used for rolled beams greater than 3'-0" [0.915 m] high and plate girders. Cross frames shall be at least 3/4 of the girder depth. For details, refer to standards developed by the FHWA Mid-Atlantic States Structural Committee for Economical Fabrication.

Diaphragm spacing shall be determined by the method specified in Section 6.7.4, Diaphragms and Cross-Frames, in the *AASHTO Specifications*.

5.4.3.8 Field Connections

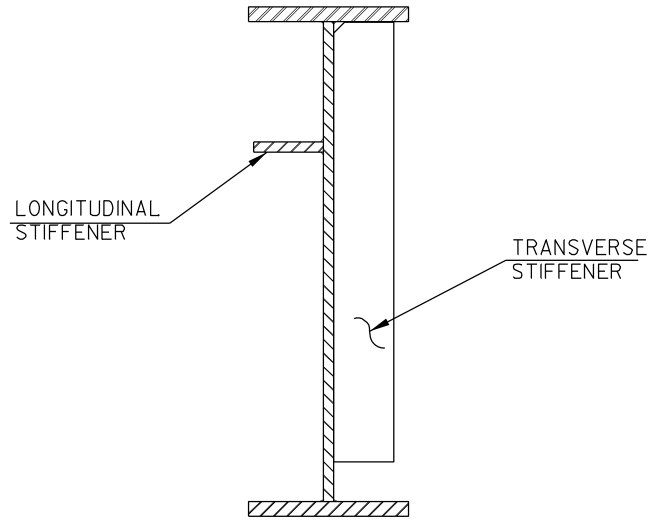
All field splices of beams or girders and diaphragm connections will be bolted. The number of splices is influenced by the length and depth of beam that can be transported to the site. Larger beams may be used at sites with access to water because barges can transport larger beams than rail or trucks. Construction considerations include site conditions and the weight of the beam. See the Section 6.13, Connections and Splices, in the *AASHTO Specifications*.

Normally, 0.875 in [22 mm] diameter, M164 [M164M] bolts are specified for field connections. In some circumstances, higher strength M253 [M253M] bolts may be considered to avoid an excessive number of bolts in splices or connections. The use of M253 [M253M] bolts must be approved by the Bridge Design Engineer. All bolts in a bridge should be the same diameter. Avoid bolts over 1 in [25 mm] in diameter.

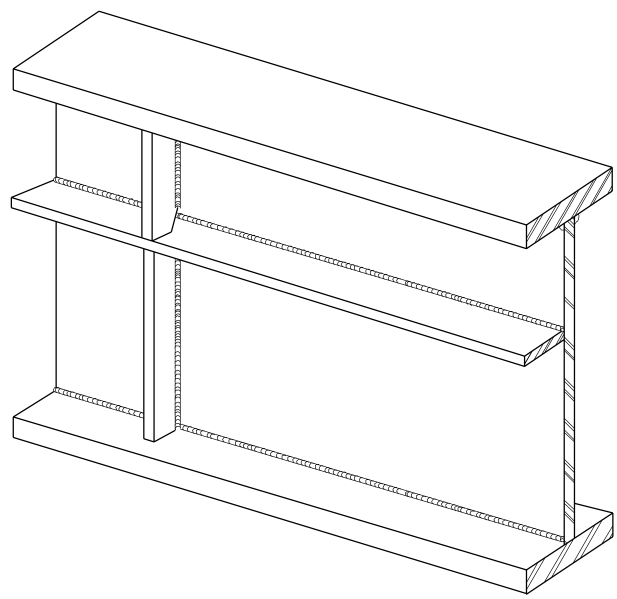
Two types of high-strength bolts are commonly used by the Department: Type 1 and Type 3. Type 1 bolts are made from medium carbon steel. Type 3 bolts have atmospheric corrosion and weathering-resistant characteristics comparable to weathering steel. Both Type 1 and Type 3 bolts are available in 0.5 in [12.7 mm] to 1.5 in [38.1 mm] diameters.

Type 1 M164 [M164M] bolts are mechanically galvanized, and painted after installation. Hot-dip galvanizing is not permitted. Type 3 M164 [M164M] bolts and Type 1 and Type 3 M253 [M253M] bolts are not galvanized, but are painted after installation.

Figure 5-27a
Longitudinal/Transverse Stiffener Intersection Details – Steel Beams

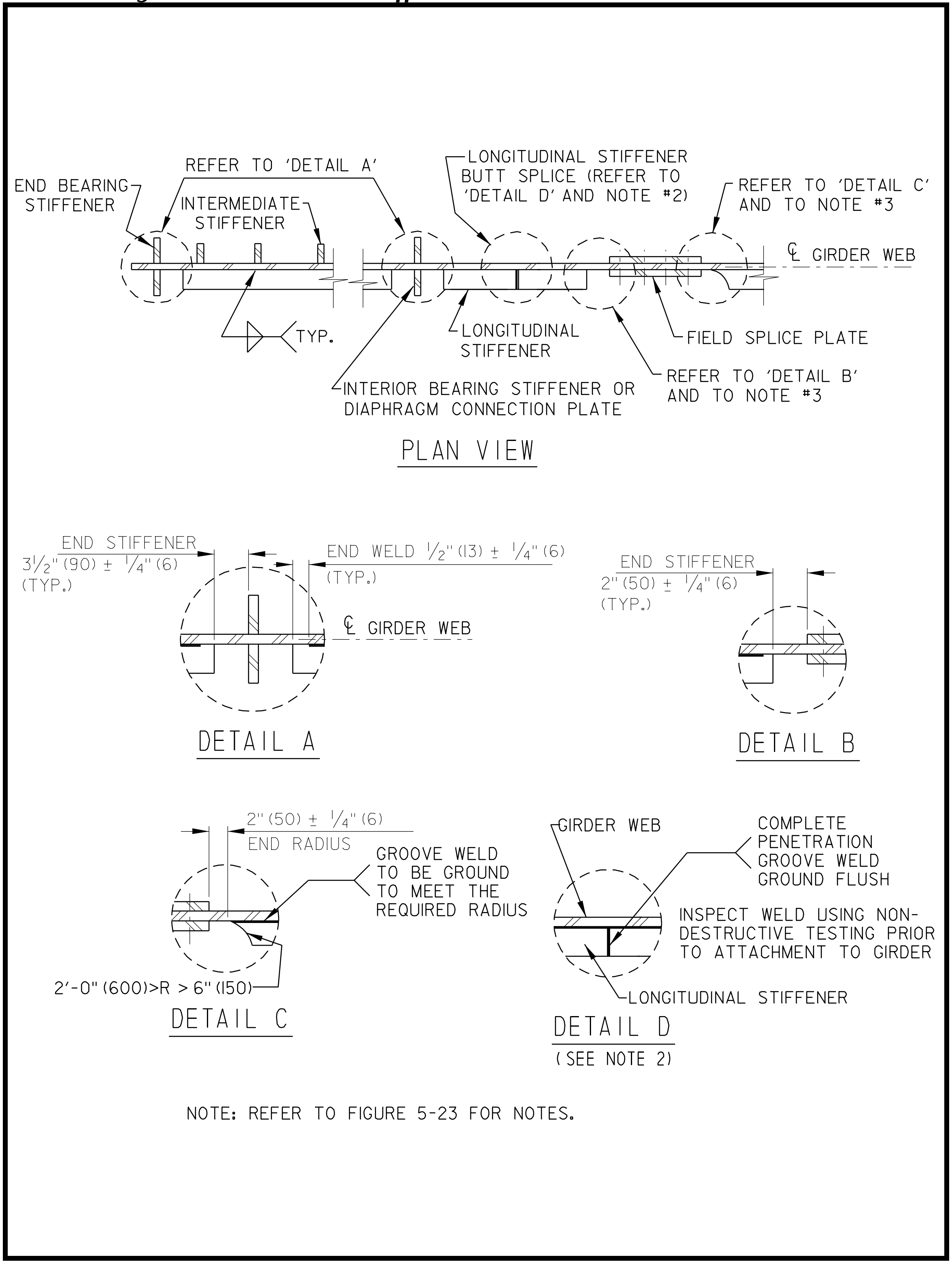


LONGITUDINAL STIFFENER ON SIDE OF WEB OPPOSITE TRANSVERSE STIFFENER.



ACCEPTABLE DETAIL FOR TRANSVERSE AND LONGITUDINAL STIFFENER ON A COMMON SIDE OF THE WEB.

Figure 5-27b
Longitudinal/Transverse Stiffener Intersection Details – Steel Beams



Splice plate thickness should be increased where longitudinal web stiffeners are terminated at bolted field splices. Thicker webs can be used to eliminate the need for longitudinal stiffeners.

5.4.4 CAMBER DIAGRAMS

Beams must be cambered in the fabrication process. A camber diagram is needed for proper fabrication of the beam and must be included in the bridge plans. A typical camber diagram is shown in Figure 5-28. Camber deflections should be computed for each beam at the 1/10th points of each span or at 10 foot [3 m] intervals, whichever is less, the same as for finished deck elevations. (See Section 5.2.4.)

The designer should furnish camber deflections for the following loadings:

- dead load due to weight of structural steel,
- dead load due to stay-in-place forms, deck reinforcing steel, and concrete deck,
- dead load due to superimposed dead loads such as wearing surfaces, sidewalks, parapets, and utilities,
- camber for vertical curve ordinate to meet proposed roadway profile, and
- total of dead load deflections and camber.

In developing camber diagrams, the designer should consider the differences in loadings, such as the effects of sidewalks, parapets, and barriers, on individual beams and girders. The deflections caused by the dead load from the structural steel, forms, and reinforced concrete deck are resisted by the steel superstructure. Deflections caused by superimposed dead load are resisted by the composite section comprised of the reinforced concrete deck and the superstructure. The fascia beam likely will not deflect the same as interior beams. Consequently, a camber diagram must be provided for fascia beams as well as for interior beams. Because the screed rail for the deck finishing machine is set from the fascia beam, camber of the fascia beam is critical to achieve the correct deck elevations, the specified deck thickness and proper drainage.

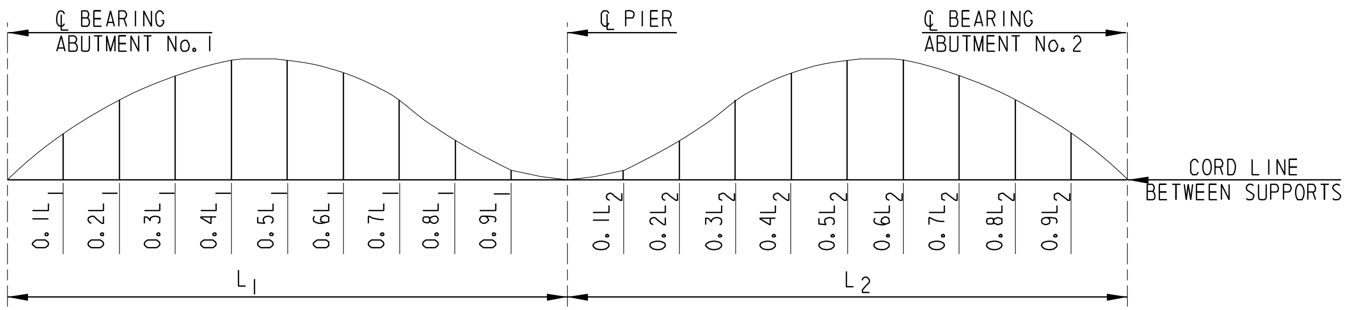
Because of the potential hazard from ponding and freezing of water on the deck, the designer must evaluate beam deflections, deck cross slope and roadway geometry as well as scupper locations to ensure that water drains properly.

5.4.5 PROTECTIVE COATINGS

The designer must provide for painting all structural steel, except weathering steel. Refer to Section 605 of the *Standard Specifications*. Normally, green, Standard Color No. 24172, Federal Standard No. 595B, is used. The use of other colors requires approval from the Assistant Director, Design, with documentation as to the reasons for the change.

It may be desirable to paint the ends of weathering steel girders near bearing areas under joints. Normally, the length of the painted area is 1.5 times the depth of the beam. Where weathering steel is painted, brown Standard Color No. 10076, Federal Standard No. 595B, is used.

Figure 5-28a
Typical Camber Diagram for Steel Beams (U.S. Customary)



DEFLECTION AND TOTAL CAMBERS INCHES

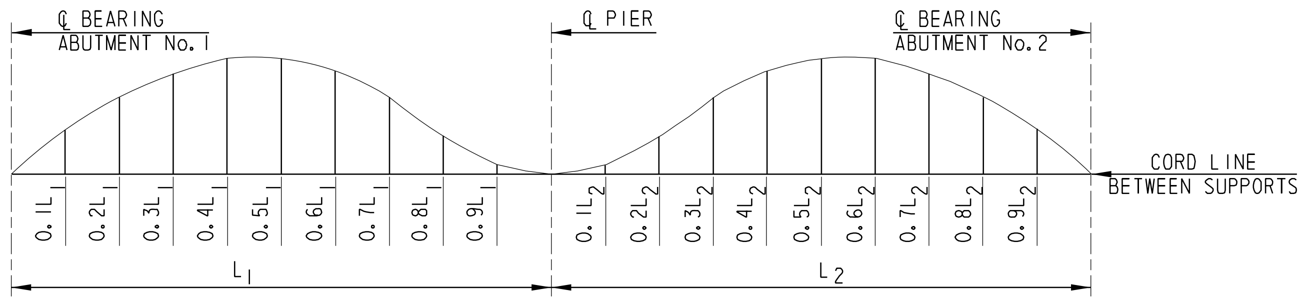
GIRDER		$0.1L_1$	$0.2L_1$	$0.3L_1$	$0.4L_1$	$0.5L_1$	$0.6L_1$	$0.7L_1$	$0.8L_1$	$0.9L_1$	℄ PIER	$0.1L_2$	$0.2L_2$	$0.3L_2$	$0.4L_2$	$0.5L_2$	$0.6L_2$	$0.7L_2$	$0.8L_2$	$0.9L_2$
FASCIA GIRDER	DLS	3/8	11/16	7/8	15/16	7/8	11/16	1/2	1/4	1/16	0	1/16	1/4	1/2	11/16	7/8	15/16	7/8	11/16	3/8
	DLC	1 13/16	3 1/4	4 1/8	4 3/8	4	3 3/16	2 1/16	1	1/4	0	1/4	1	2 1/16	3 3/16	4	4 3/8	4 1/8	3 1/4	1 13/16
	SDL	1/4	7/16	9/16	5/8	9/16	1/2	5/16	3/16	1/16	0	1/16	3/16	5/16	1/2	9/16	5/8	9/16	7/16	1/4
	TD&C	2 7/16	4 7/16	5 5/8	5 15/16	5 1/2	4 3/8	2 7/8	1 7/16	3/8	0	3/8	1 7/16	2 7/8	4 3/8	5 1/2	5 15/16	5 5/8	4 7/16	2 7/16
	VCO	1/4	7/16	9/16	11/16	11/16	11/16	9/16	7/16	1/4	0	1/4	7/16	9/16	11/16	11/16	11/16	9/16	7/16	1/4
	TD&C +VCO	2 11/16	4 7/8	6 3/16	6 5/8	6 3/16	5 1/16	3 7/16	1 7/8	5/8	0	5/8	1 7/8	3 7/16	5 1/16	6 3/16	6 5/8	6 3/16	4 7/8	2 11/16
INTERIOR GIRDER	DLS	3/8	11/16	7/8	15/16	7/8	11/16	7/16	1/4	1/16	0	1/16	1/4	7/16	11/16	7/8	15/16	7/8	11/16	3/8
	DLC	1 7/16	2 9/16	3 1/4	3 7/16	3 1/8	2 1/2	1 5/8	13/16	3/16	0	3/16	13/16	1 5/8	2 1/2	3 1/8	3 7/16	3 1/4	2 9/16	1 7/16
	SDL	1/4	3/8	1/2	9/16	1/2	7/16	5/16	1/8	1/16	0	1/16	1/8	5/16	7/16	1/2	9/16	1/2	3/8	1/4
	TD&C	2	3 5/8	4 5/8	4 15/16	4 1/2	3 5/8	2 3/8	1 3/16	5/16	0	5/16	1 3/16	2 3/8	3 5/8	4 1/2	4 15/16	4 5/8	3 5/8	2
	VCO	1/4	7/16	9/16	11/16	11/16	11/16	9/16	7/16	1/4	0	1/4	7/16	9/16	11/16	11/16	11/16	9/16	7/16	1/4
	TD&C +VCO	2 1/4	4 1/16	5 3/16	5 9/16	5 1/4	4 1/4	2 15/16	1 5/8	5/16	0	9/16	1 5/8	2 15/16	4 1/4	5 1/4	5 9/16	5 3/16	4 1/16	2 1/4

LEGEND

DLS - DENOTES DEFLECTION DUE TO STRUCTURAL STEEL
DLC - DENOTES DEFLECTION DUE TO CONCRETE SLAB
SDL - DENOTES DEFLECTION DUE TO PARAPET
TD&C - DENOTES TOTAL DEAD LOAD DEFLECTION AND CAMBER
VCO - DENOTES CAMBER FOR VERTICAL CURVE ORDINATE DUE TO ROADWAY PROFILE

NOTE: ALL DIMENSIONS ARE IN INCHES, UNLESS NOTED.

Figure 5-28b
Typical Camber Diagram for Steel Beams [Metric]



DEFLECTION AND TOTAL CAMBERS (mm)

GIRDER		$0.1L_1$	$0.2L_1$	$0.3L_1$	$0.4L_1$	$0.5L_1$	$0.6L_1$	$0.7L_1$	$0.8L_1$	$0.9L_1$	℄ PIER	$0.1L_2$	$0.2L_2$	$0.3L_2$	$0.4L_2$	$0.5L_2$	$0.6L_2$	$0.7L_2$	$0.8L_2$	$0.9L_2$
FASCIA GIRDER	DLS	10	17	22	24	22	17	13	6	2	0	2	6	13	17	22	24	22	17	10
	DLC	46	82	105	111	102	81	52	25	6	0	6	25	52	81	102	111	105	82	46
	SDL	6	11	14	16	14	13	8	5	2	0	2	5	8	13	14	16	14	11	6
	TD&C	62	113	143	151	140	111	73	36	10	0	10	36	73	111	140	151	143	113	62
	VCO	6	11	14	17	17	17	14	11	6	0	6	11	14	17	17	17	14	11	6
	TD&C +VCO	68	124	157	168	157	129	87	48	16	0	16	48	87	129	157	168	157	124	68
INTERIOR GIRDER	DLS	10	17	22	24	22	17	11	6	2	0	2	6	11	17	22	24	22	17	10
	DLC	36	65	82	87	79	63	41	21	5	0	5	21	41	63	79	87	82	65	36
	SDL	6	10	13	143	13	11	8	3	2	0	2	3	8	11	13	143	13	10	6
	TD&C	51	92	117	125	114	92	60	30	8	0	8	30	60	92	114	125	17	92	51
	VCO	6	11	143	17	17	17	14	11	6	0	6	11	14	17	17	17	143	11	6
	TD&C +VCO	57	102	132	141	133	108	75	41	14	0	14	41	75	108	133	141	132	101	57

LEGEND

DLS - DENOTES DEFLECTION DUE TO STRUCTURAL STEEL
DLC - DENOTES DEFLECTION DUE TO CONCRETE SLAB
SDL - DENOTES DEFLECTION DUE TO PARAPET
TD&C - DENOTES TOTAL DEAD LOAD DEFLECTION AND CAMBER
VCO - DENOTES CAMBER FOR VERTICAL CURVE ORDINATE DUE TO ROADWAY PROFILE

NOTE: ALL DIMENSIONS ARE IN MILLIMETERS, UNLESS NOTED.

5.5 PRESTRESSED CONCRETE BEAMS

All concrete beams for bridges will be precast and prestressed by pretensioning. Post-tensioning may be justified for prestressing on a case-by-case basis.

5.5.1 TYPES

Delaware uses these types of precast-prestressed concrete beams:

- voided slabs,
- AASHTO I-girders,
- adjacent or spread box girders, and
- Bulb-T Girders.

AASHTO, in conjunction with the Precast/Prestressed Concrete Institute (PCI), has developed several standard voided slabs, I and box sections commonly referred to as the AASHTO Girders or the PCI-AASHTO Standard Sections.

Voided or solid slabs. The AASHTO voided slab is commonly used for short spans ranging from 30 to 50 feet [9.1 to 15 m]. AASHTO has standardized five sections to accommodate any combination of bridge width and span length in the 30 to 50 feet [9.1 to 15 m] range. The sections are 3'-0" or 4'-0" [0.9 or 1.2 m] wide with depths of 15, 18, and 21 inches [381, 457 and 533 mm]. Thinner sections may be designed without voids.

AASHTO I-Girders. These girders are generally used for short to intermediate spans. They can be modified to accommodate longer spans.

Adjacent and spread box girders. For spans greater than 50 feet [15.2 m] AASHTO has also developed a series of standard box sections. Like the voided slabs, standard sections are available in 3'-0" and 4'-0" [0.9 m and 1.2 m] widths and a variety of depths to accommodate various bridge widths and span lengths.

Bulb-T Girders. The FHWA Mid-Atlantic States Prestressed Concrete Committee for Economic Fabrication (PCEF) has developed a series of Bulb-T girders that offer a wide range of girder depths, flange widths, and web widths. These girders generally provide a more economical use of materials than the AASHTO I-Girders. The designer should use PCEF shapes.

5.5.2 MATERIALS

Concrete with an f'_c = 5,000 psi [35 MPa] is normally used for prestressed concrete beams. An increase to a higher strength is permissible when approved by the Bridge Design Engineer.

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified. The minimum size of reinforcing is a #4 [13] bar.

Normally, prestressing strands shall be high-strength 7-wire low-relaxation strand, with nominal 0.5 inch [12.7 mm] or 0.6 inch [15.2 mm] diameter, and conform to AASHTO M203, 270 ksi [1862 MPa] grade, low-relaxation strands.

On post-tensioned structures, the designer will specify that all strands will be uncoated and all strand conduits will be pressure-grouted.

5.5.3 DESIGN

Prestressed concrete beams will be designed as simply supported for dead load and continuous for live load. Refer to Section 5, Concrete Structures, in the *AASHTO Specifications*.

5.5.3.1 Beam Spacing

The Department has adopted the following spacings for concrete beams:

- Desirable - 9'-0" [2.7 m]
- Maximum - 10'-0" [3.0 m]

These spacings do not apply to adjacent box girders. Where vertical clearance is not a problem, a wider maximum spacing (up to 12'-6" [3.8 m]) may be justified, on a case-by-case basis.

The width of the top flange of Type 5 or 6 AASHTO girders and Bulb-T girders leaves little unsupported deck. Where these girders are used, the maximum beam spacing may be increased or the thickness of the deck may be decreased to the minimum thickness specified in Section 5.2.1.3.

5.5.3.2 Composite Design

Composite flexural members consist of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to superimposed loads as a unit. The entire composite member may be used in resisting shear and moment. Refer to Section 5, Concrete Structures, in the *AASHTO Specifications*. See Figure 5-29 for a typical composite box beam.

Cast-in-place deck slabs are generally designed to provide composite action with concrete box or I-girders. Precast deck slabs can be designed to provide composite action with concrete I-girders if block-outs are included in the slab and shear transfer connections are provided. In this case, a quick-setting, non-shrink concrete is placed in the blocked-out area.

The use of composite designs is recommended and must be noted under Project Notes on the plans for future reference.

5.5.3.3 Shear Developers

Bar reinforcing is used to provide resistance to horizontal shear between the deck and the girder. See Section 5.8.4, Interface Shear Transfer—Shear Friction, in the *AASHTO Specifications*. Shear developer details for adjacent concrete box girders and I-girders are shown in Figure 5-29.

Shear between adjacent concrete box girders or voided slabs can be provided by shear connections or keys. Refer to Figure 5-30. Grout and shear key shall reach a minimum compressive strength of 5 ksi [35 MPa] in 24 hours as per Section 5.14.4.3.2 of the *AASHTO Specifications.*

Figure 5-29
Composite Details for Concrete Beams

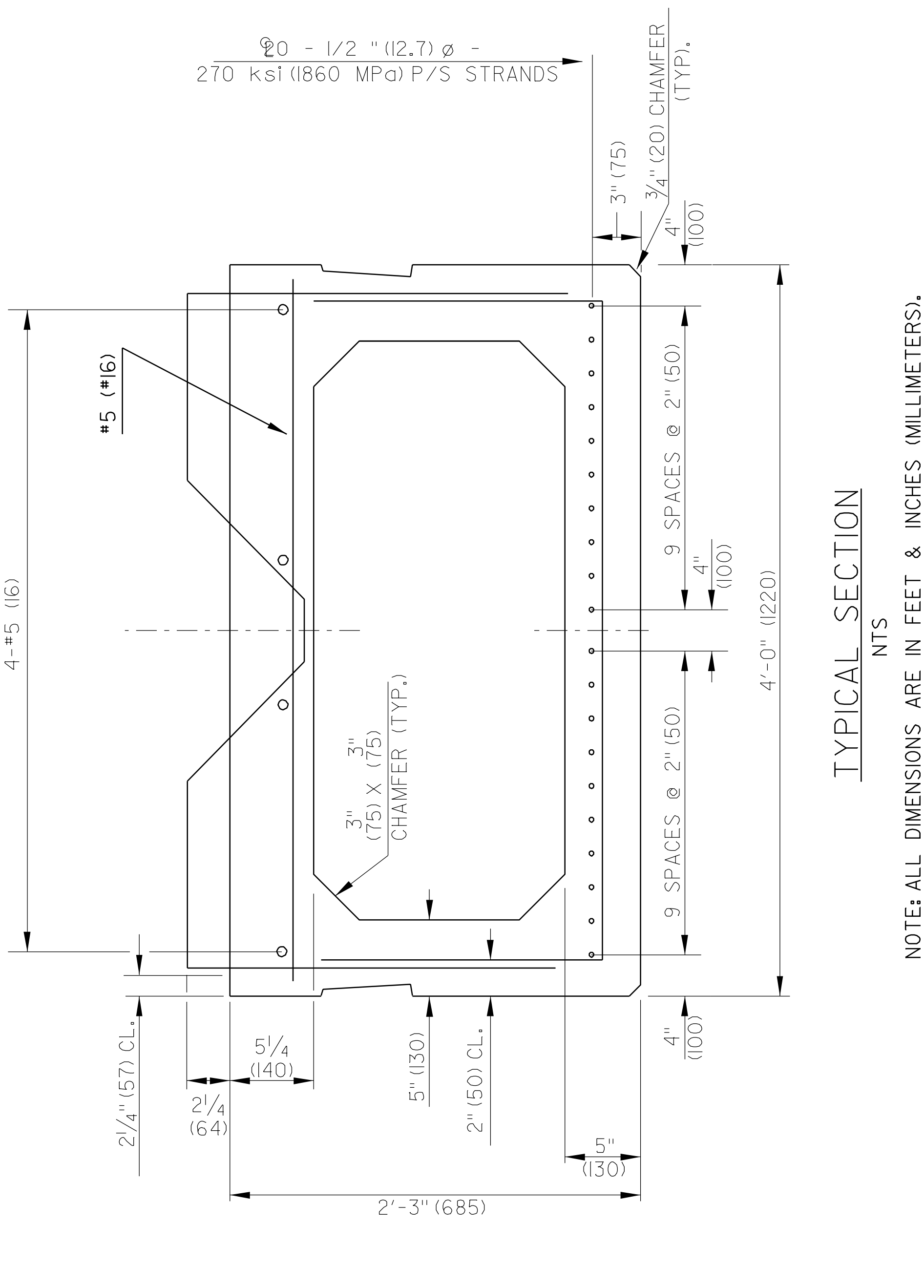
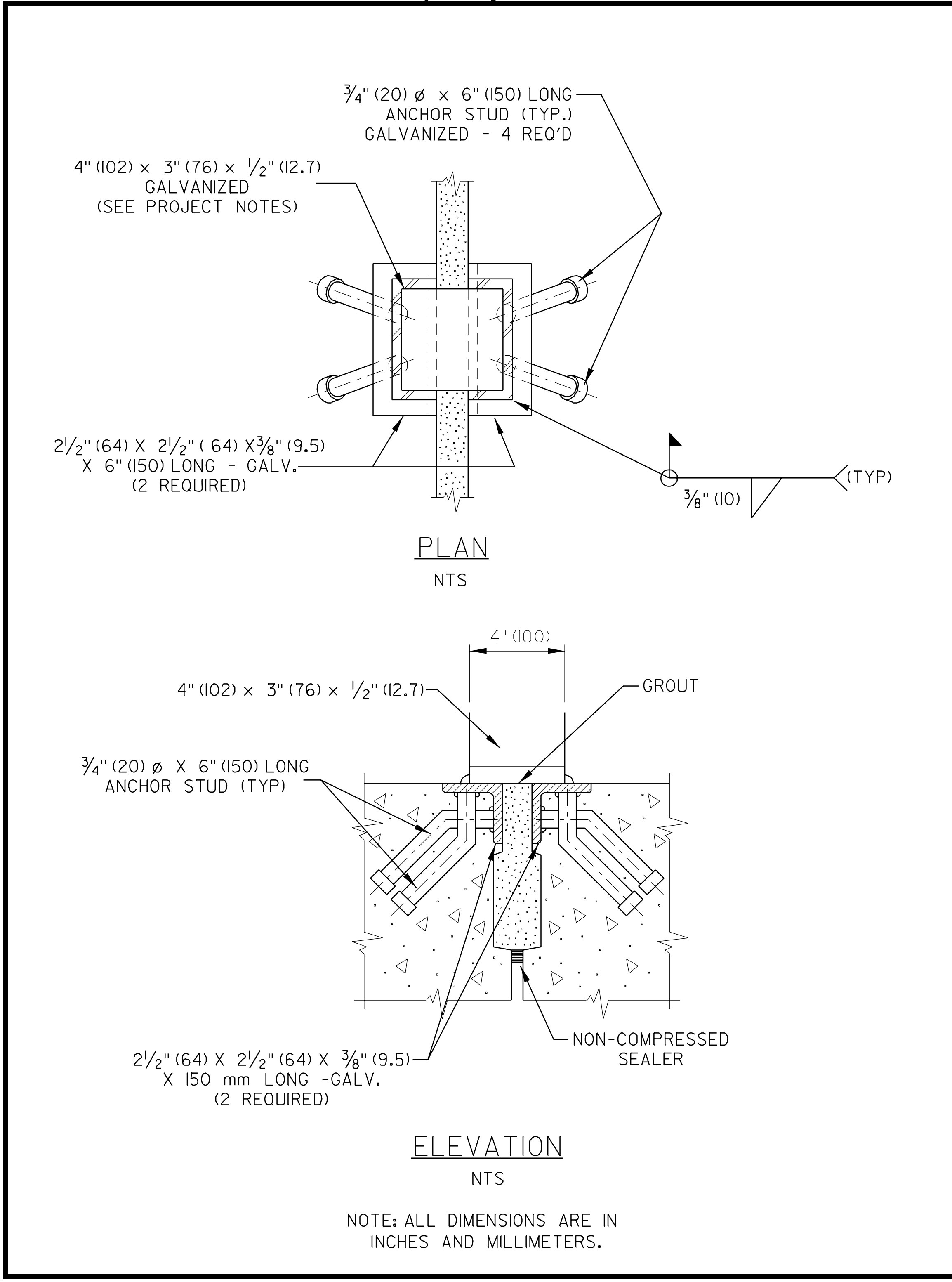


Figure 5-30
Shear Connection Details for Adjacent Concrete Box Girders



5.5.3.4 Continuity for Live Load

Concrete I-girders are designed as simple spans for dead load but as continuous beams for live load. Continuity is attained by designing additional reinforcing steel in the deck over beam joints for the negative moments and with monolithically poured reinforced diaphragms at the beam ends.

5.5.3.5 Diaphragms

The end diaphragms for each span on skewed bridges must also be skewed. Intermediate diaphragms are perpendicular to the beams. Diaphragms are poured and cured prior to pouring the deck. The minimum number of diaphragms is three per span: one at each end and one at midspan.

5.5.3.6 Strand Design

All prestressing strands for prestressed beams will be located inside the stirrups. See Section 5.10.3.3, Minimum Spacing of Prestressing Tendons and Ducts, in the *AASHTO Specifications*. The center of gravity of the strands should be shown on the plans.

5.5.3.7 Camber Diagram

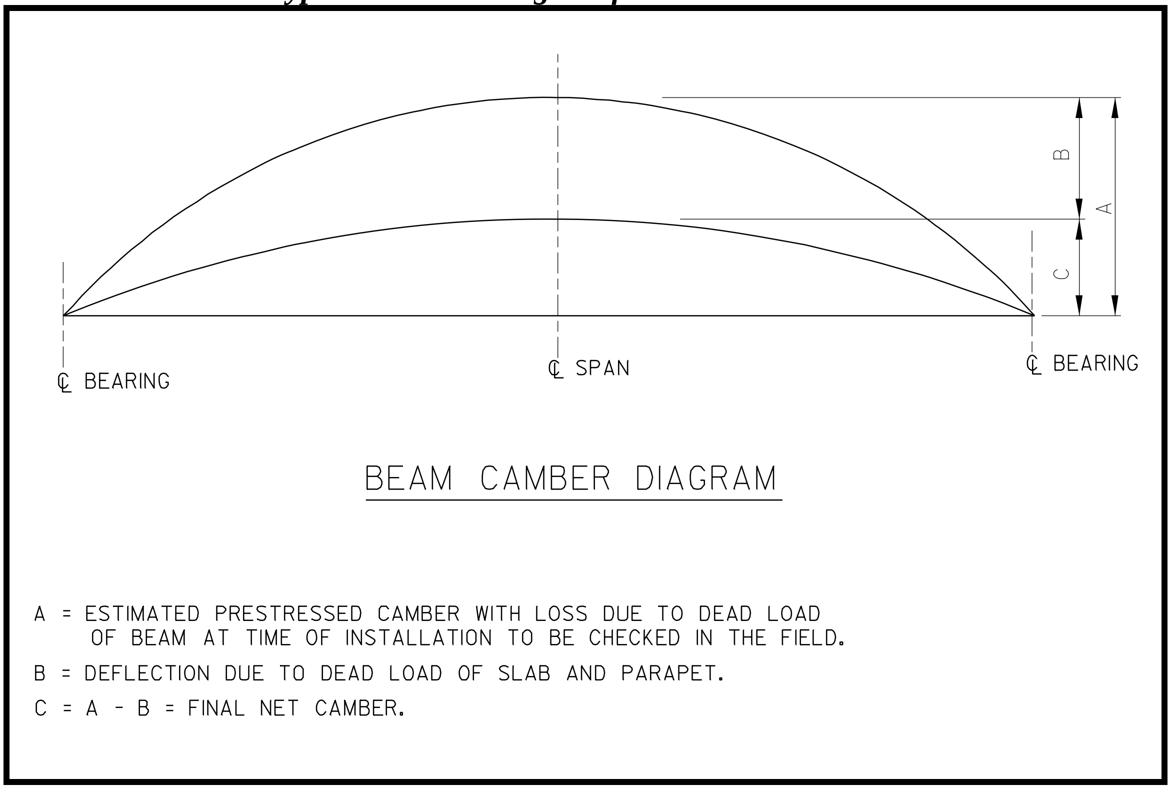
Camber in prestressed beams is a function of prestressing and dead load, and needs to be verified in the field. A camber diagram must be included in the bridge plans. Figure 5-31 shows a typical camber diagram. Camber deflections should be computed for each beam at the midpoint of each span. The designer should furnish camber deflections for the following:

- The estimated prestress camber loss due to the dead load of the beam at the time of installation multiplied by a creep factor.
- The deflection due to dead load of the slab and parapet.
- The final net camber, which is the combination of the first two.

The above are theoretical values and may vary with actual concrete strength (age), various prestressing conditions, creep, and prestress loss. Creep of concrete is the time-dependent deformation of concrete under a sustained load. No creep factor is assumed in calculating dead load deflection for the slab and parapet. See Precast/Prestressed Concrete Institute's *Design Supplement to Precast Prestressed Concrete*, *Short Span Bridges, Spans to 100 Feet*.

In developing camber diagrams, the designer should consider the differences in loadings, such as the effects of sidewalks, parapets, and barriers, on individual beams and girders. The deflections caused by the dead load from the concrete girders, forms, and reinforced concrete deck are resisted by the concrete superstructure. Deflections caused by superimposed dead load are resisted by the composite section comprised of the reinforced concrete deck and the superstructure. The fascia beam likely will not deflect the same as interior beams. Consequently, a camber diagram must be provided for fascia beams as well as for interior beams. Because the screed rail for the deck finishing machine is set from the fascia beam, camber of the fascia beam is critical to achieve the correct deck elevations, the specified deck thickness and proper drainage.

Figure 5-31
Typical Camber Diagram for Concrete Beams



Because of the potential hazard from ponding and freezing of water on the deck, the designer must evaluate beam deflections, deck cross slope and roadway geometry as well as scupper locations to ensure that water drains properly.

5.6 BEARINGS

5.6.1 BEARING DEVICES

Bearing devices are designed to transmit loads from the superstructure to the substructure and to provide for expansion, contraction, and rotation of the superstructure. The devices must be able to withstand forces from several directions simultaneously. The bearings must also accommodate movements of the structure that result from loads, temperature change, deflection, impact and centrifugal force. The bearings should be designed to be easy to maintain with minimal maintenance. Consideration should also be given to the future need to jack girders to permit repair, lubrication, and maintenance of bearing devices.

There are two types of bearings: fixed and expansion. Some bearing materials can be used for either fixed or expansion bearings. The amount of movement that each type of bearing can provide must be considered in the selection. The types of bearings include:

- reinforced elastomeric pads,
- unreinforced elastomeric pads,
- steel sliding plate for expansion,
- steel plate for fixed bearing,
- high-load multi-rotational, and
- pot.

Refer to Section 14, Joints and Bearings, in the *AASHTO Specifications* for design requirements. Figure 5-32 provides notes for interpreting general information on bearings shown in Figures 5-33 through 5-38.

Reinforced elastomeric bearings are a combination of materials that may include reinforced fiber mesh, steel and neoprene. They require the least maintenance but are susceptible to deterioration from ozone and ultraviolet light. Reinforced elastomeric bearings are suitable for movements up to 2.5 in [65 mm]. Reinforced elastomeric bearings should be considered over other types of bearings in the majority of cases. See Figure 5-33.

Elastomeric pads are unreinforced single material bearing pads. They are used at fixed ends of voided slab and concrete box girder structures. See Figure 5-34.

Steel sliding plate bearings (expansion) may be a combination of steel plate, polished stainless steel sheet, polytetrafluoroethelyene (TFE), and urethane. Steel sliding plate bearings are suitable for movements up to 2 in [50 mm]. These bearings are less vulnerable to environmental deterioration than elastomeric bearings. Longitudinal movement is accommodated by either two polished surfaces sliding on each other or the stainless steel sliding on the TFE surface. Rotation is provided by the curved surfaces sliding on each other or by deformation of the urethane pad on TFE bearings. See Figure 5-35.

Fixed steel plate bearings anchor the span and provide for rotation of the fixed end of the beam. Rotation is provided by the curved surfaces sliding on each other. See Figure 5-36. The height of the fixed bearing assembly can be designed to accommodate various bearing seat and beam requirements.

Rotational bearings are made with matching machined steel concave and convex surfaces. Rotational bearings may be either fixed or expansion. Rotation is accommodated at the matched concave and convex surfaces. Longitudinal movement for the expansion bearing is provided by sliding of the concave plate against the sole plate of the beam. They require a minimum of maintenance. Rotational expansion bearings are suitable for movements up to 2.5 in [65 mm]. See Figure 5-37.

Pot bearings are high-load multi-rotational bearings. The basic rotational bearing can be combined with a TFE and stainless steel sheet to allow translation. The direction of translation can be controlled using guide bars. Pot bearings consist of a piston and pot arrangement similar to a hydraulic cylinder in which elastomer is used to accommodate the rotational deformation. Inherent problems with this type of bearing include leakage of the elastomer and failure of the seal rings. See Figure 5-38.

Rocker bearings (which are not used by the Department) are mechanical-type bearings which can accommodate translation and rotation in only one direction. Rocker bearings are not used by the Department because of their increased seismic vulnerability and high maintenance requirements.

Figure 5-32a
Notes for Bearings

GENERAL NOTES FOR ALL BEARINGS

1. Design is based on AASHTO M183 (ASTM A36) steel stresses.
2. T1, T3, Ts, Tb and y are measured at centerline of bearing at 70°F (21°C).
3. Fill slots and holes in masonry plate around anchor bolts with an approved non-hardening caulking compound or elastic joint sealer.
4. Steel surfaces of sole plate, rocker plate, web, and bearing plate to be a machine finish as shown in the details, measured in accordance with ANSI B46.1. Other steel plate surfaces to be finished to at least 1 in [25 mm].
5. Bearing details are satisfactory for the E. Q. requirement for Zone 1.
6. Bearings shall be shop assembled and match marked to ensure proper fit, if uplift is present.
7. Anchor bolts, where required, shall meet the requirements of AASHTO M183, and shall have hex nuts and washers. When an anchor bolt goes though the sole plate, the nut is to be 0.25 in [6 mm] clear. Burr threads at face of nut.
8. Anchor bolts shall be swedged and may be cast in place or grouted in preformed (sleeved or drilled) holes.
9. Sole plates are to be beveled or have radii machined to match grade when grade exceeds 3 percent for low-profile fixed bearings or 1 percent for all other bearings.
10. Steel plates shall meet a flatness requirement of 0.5 percent in the direction being measured (width, length, and diagonals) maximum, but not to exceed 1/8 in [3 mm].
11. For painted structures, metal bearings shall be coated with one shop coat of paint in accordance with project requirements.
12. A leveling pad shall be placed under the masonry plate where required by the contract.

NOTES FOR EXPANSION BEARINGS — FIGURE 5-35

13. Δ = the total longitudinal movement the bearing can take. If calculated movement exceeds the limit shown, plate widths may be increased or the next higher capacity bearing may be substituted.
14. Surfaces of sole plates and masonry plates in contact with bronze plates are to have a machined finish of at least 1/8 in [3 mm]. These steel surfaces shall be coated with a multipurpose grease before shipment—not painted. The coating shall be removed with a solvent immediately prior to erection.

Figure 5-32b
Notes for Bearings

15. Self-lubricating bronze bearing plates shall conform to the requirements of ASTM B22, copper alloy UNS C90500 modified with up to 2V2 percent lead maximum. The sliding surfaces of the plates shall be provided with annular grooves or cylinder recesses or a combination thereof, which will be filled with a lubricating compound. The lubricating compound shall be compressed into the recesses under sufficient pressure to form a nonplastic lubricating inset. The lubricating inset shall comprise not less than 25 percent of the total area of the plate. The frictional coefficient shall not exceed 0.1, when the bearing is tested under a load of R and for 1,000 cycles. The compound shall be free of any material that could cause abrasive or corrosive action upon the metal surfaces and also shall be able to withstand extremely high pressures and the atmospheric elements over long periods of time.

    All items shall be the standard products of the manufacturer of such materials for this application.

    Prior to assembly in place, the steel surface which will bear on the self-lubricating bearing plate shall be thoroughly lubricated with additional antioxidant lubricant furnished by the manufacturer.

Figure 5-33a
Elastomeric Bearing Details

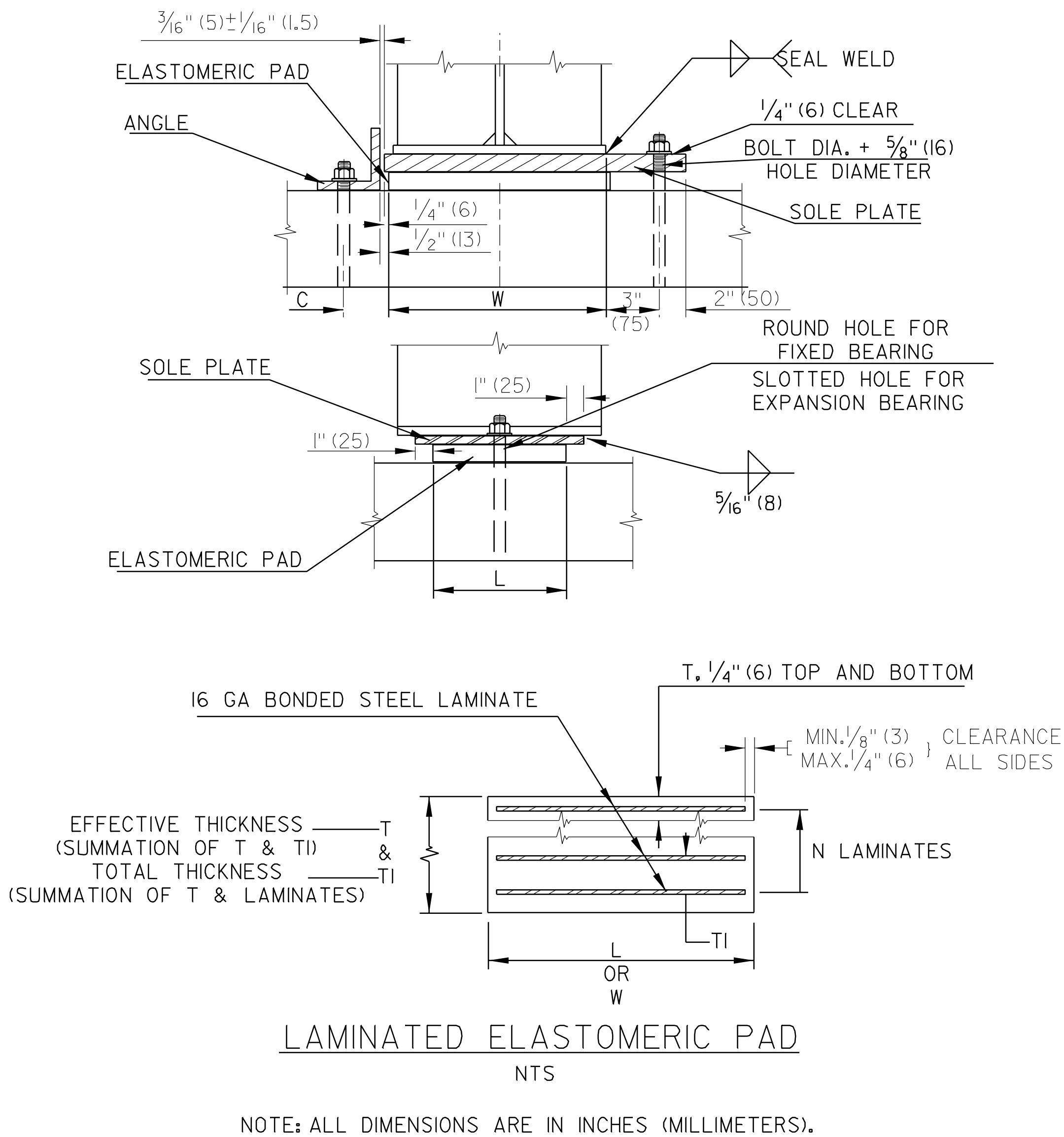


Figure 5-33b
Elastomeric Bearing Details

ENGLISH

TYPE	SPAN FT.	R TONS	W"	L"	t"	N	T"	△ "	T1 "	C "	D "	SIZE OF ANGLE
SIMPLE SPAN	20	50	10	12	1/2	0	1/2	1/4	1/2	2 1/4	1	3 x 3 x 1/2 x 6
SIMPLE SPAN	30	60	10	8	1/2	2	1	1/2	1 1/4	2 1/4	1	3 x 3 x 1/2 x 6
SIMPLE SPAN	70	100	12	10	1/2	4	2	1	2 1/2	2 7/8	1 1/4	4 x 4 x 3/8 x 8
3 SPAN CONT.	70	140	12	12	1/2	4	2	1	2 1/2	2 7/8	1 1/4	4 x 4 x 3/8 x 8
3 SPAN CONT.	90	180	14	15	1/2	5	2 1/2	1 1/4	3 1/8	2 7/8	1 1/4	6 x 4 x 3/4 x 10
3 SPAN CONT.	100	220	14	16	1/2	5	2 1/2	1 1/4	3 1/8	2 7/8	1 1/4	6 x 4 x 3/4 x 10
3 SPAN CONT.	120	260	16	19	1/2	6	3	1 1/2	3 1/2	3 7/8	1 1/2	6 x 6 x 3/4 x 12
3 SPAN CONT.	140	300	16	22	1/2	7	3 1/2	1 3/4	4 3/8	4	1 1/2	8 x 6 x 1 x 14
3 SPAN CONT.	160	340	18	22	1/2	8	4	2	5	4	1 1/2	8 x 6 x 1 x 14
3 SPAN CONT.	200	400	20	25	1/2	10	5	2 1/2	6 1/4	4	1 1/2	8 x 6 x 1 x 16

METRIC

TYPE	SPAN (m)	R (kN)	W (mm)	L (mm)	t (mm)	N	T (mm)	△ (mm)	T1 (mm)	C (mm)	D (mm)	SIZE OF ANGLE
SIMPLE SPAN	6	220	254	305	13	0	13	6	13	57	25	76x76x12.7x152
SIMPLE SPAN	9	265	254	203	13	2	25	13	32	57	25	76x76x12.7x152
SIMPLE SPAN	21	445	305	254	13	4	51	25	64	730	32	76x76x12.7x203
3 SPAN CONT.	21	625	356	305	13	4	51	25	64	730	32	76x76x12.7x203
3 SPAN CONT.	27	800	356	381	13	5	64	32	79	730	32	76x76x12.7x254
3 SPAN CONT.	30	980	406	406	13	5	64	32	79	730	32	76x76x12.7x254
3 SPAN CONT.	37	1155	406	483	13	6	76	38	95	984	38	76x76x12.7x305
3 SPAN CONT.	43	1335	457	559	13	7	89	44	111	1016	38	76x76x12.7x356
3 SPAN CONT.	49	1510	457	559	13	8	102	51	127	1016	38	76x76x12.7x356
3 SPAN CONT.	61	1780	508	635	13	10	127	64	159	1016	38	76x76x12.7x406

NOTE: THIS TABLE SHALL ONLY BE USED FOR PRELIMINARY SIZING. ACTUAL DESIGN SHALL BE IN ACCORDANCE WITH AASHTO SPECIFICATIONS.

Figure 5-34
Neoprene Bearing Details

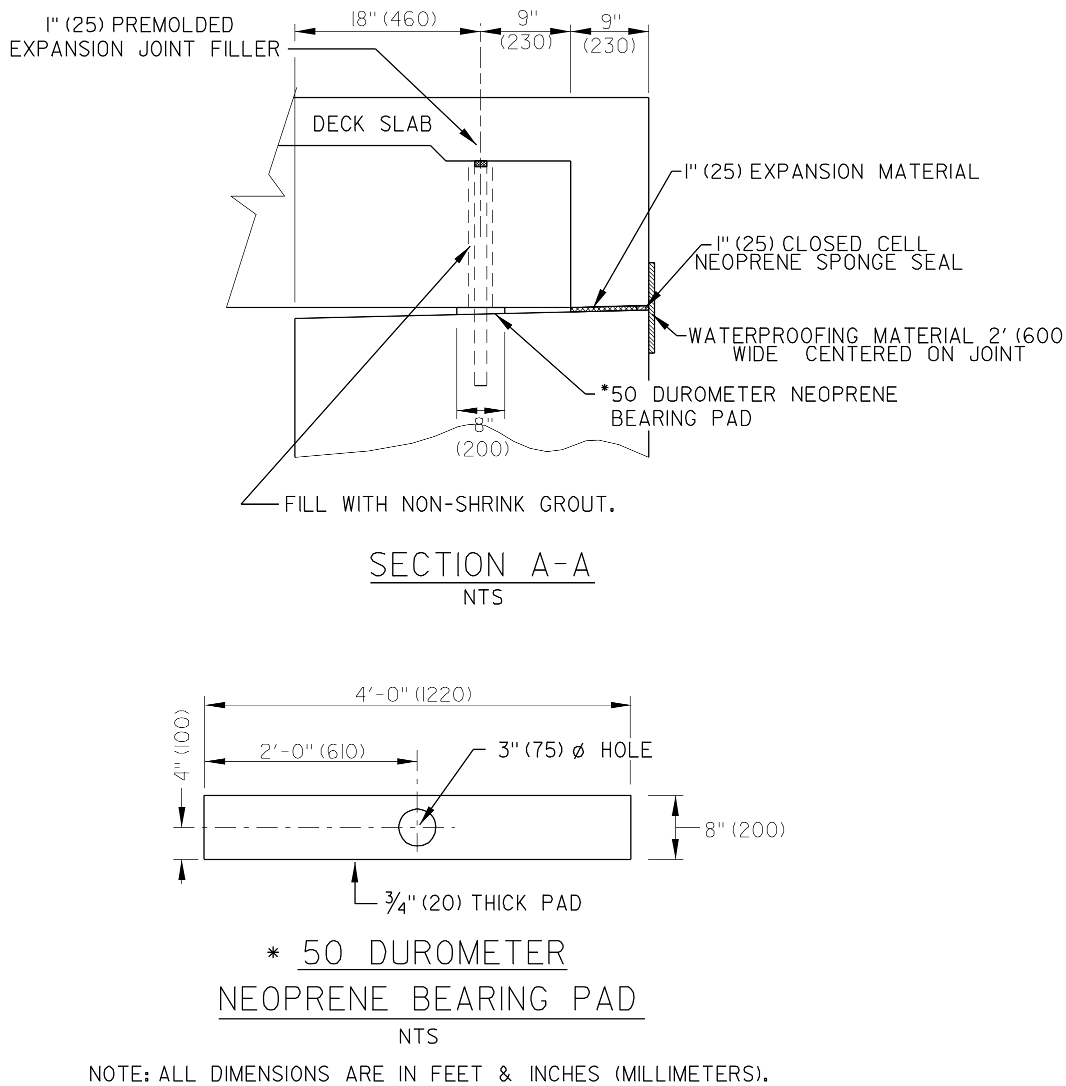
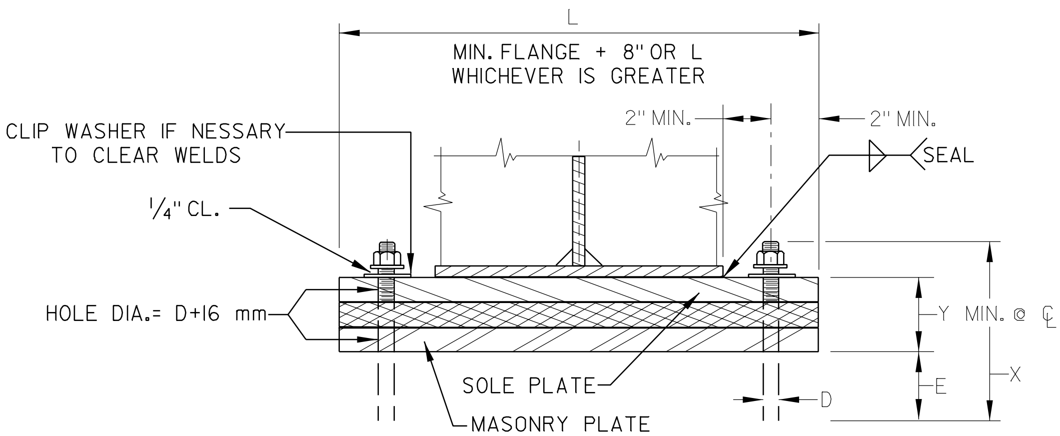


Figure 5-35a
Steel Bearings – Expansion (U.S. Customary)

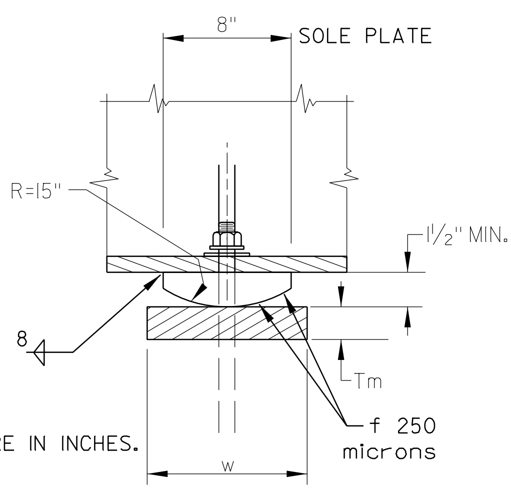
FIXED BEARING

REACTIONS (R) - UP TO 400 k



KIPS	INCHES						
R	L	W	T	D	=E	X	Y
140	20	8	1 1/2	1 1/4	12	18	3
180	22	9	1 3/4	1 1/4	12	18	3 1/4
220	24	10	2	1 1/2	15	22	3 1/2
260	26	10	2	1 1/2	15	22	3 1/2
300	28	11	2 1/4	1 1/2	15	22	3 3/4
340	30	12	2 1/2	1 1/2	15	22	4
400	32	13	2 1/2	1 1/2	15	22	4

NOTE:
SEE FIGURE 5-29

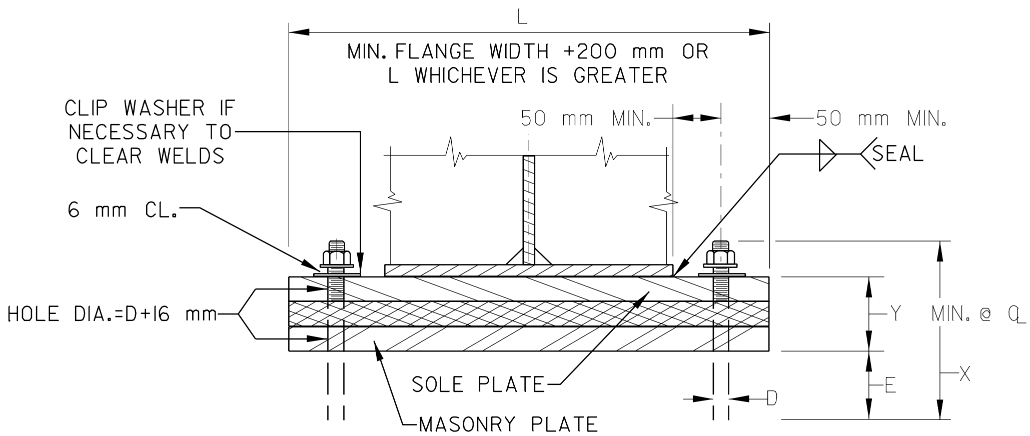


NOTE: ALL DIMENSIONS ARE IN INCHES.

Figure 5-35b
Steel Bearings – Expansion [Metric]

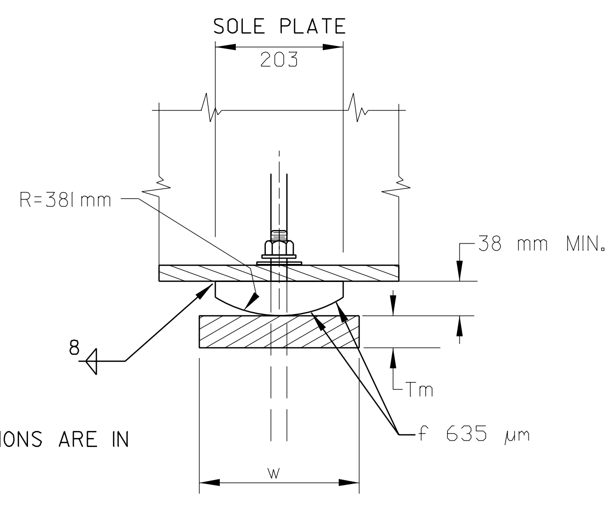
FIXED BEARING

REACTIONS (R) - UP TO 1780 kN



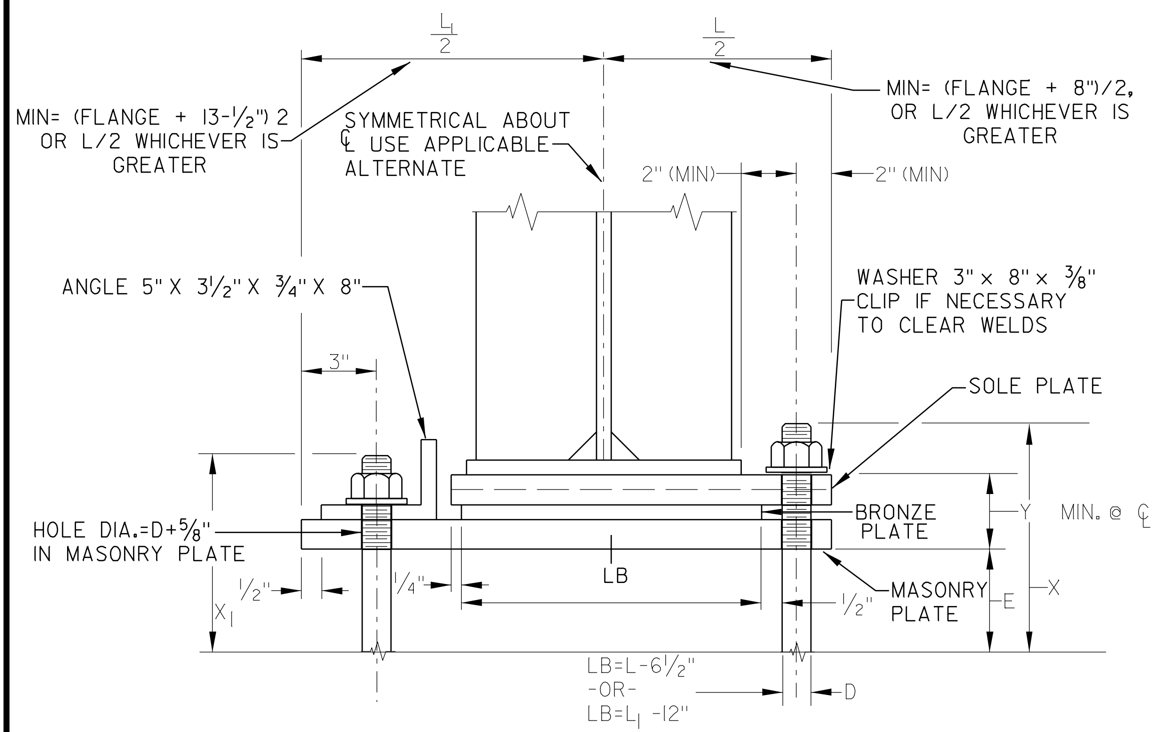
R kN	L mm	W mm	Tm	D mm	E mm	X mm	Y mm
620	508	203	38	32	305	457	76
800	559	229	44	32	305	457	83
980	610	254	51	38	381	559	89
1155	660	254	51	38	381	559	89
1335	711	279	57	38	381	59	95
1510	762	305	64	38	381	559	102
1780	813	330	64	38	381	559	102

NOTE:
SEE FIGURE 5-29



NOTE: ALL DIMENSIONS ARE IN
MILLIMETERS.

Figure 5-36a
Steel Bearings – Fixed (U.S. Customary)

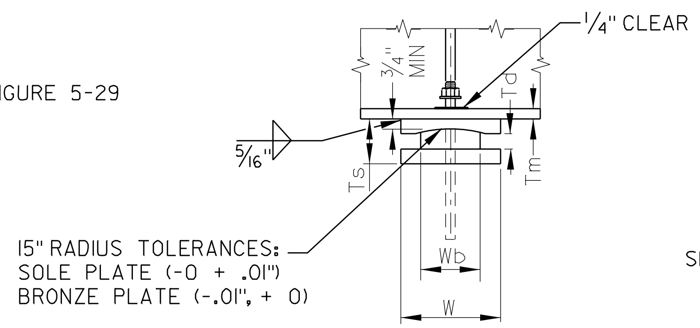


ANCHOR BOLTS THROUGH SOLE PLATES ARE ONLY NECESSARY WHEN UPLIFT COULD OCCUR. WHEN BOLTS PASS THROUGH SOLE PLATES, EXTREME CARE SHALL BE EXERCISED IN PROPERLY INSTALLING ANCHOR BOLTS.

KIPS	INCHES												
R	L	W	Wd	Ts	Tm	Tb	D	E	X	Y	△	l_1	X_1
120	20	10	8	1	2	1½	1¼	12	18¾	4¼	1	25½	17¼
180	22	12	9½	2	2¼	1½	1¼	12	19	4½	1	27½	17½
220	24	12½	10	2¼	2½	1¾	1½	15	22½	5	1½	29½	20¾
260	26	13½	11	2½	2½	2	1½	15	22¾	5¼	1½	31½	20¾
300	28	4	11	2½	2¾	2	1½	15	23	5½	2	33½	21
340	30	14½	11½	2¾	2¾	2	1½	15	23	5½	2	35½	21
400	32	15½	12½	3	3	2¼	1½	15	23½	6	2	37½	21¼

R L

NOTE:
SEE FIGURE 5-29

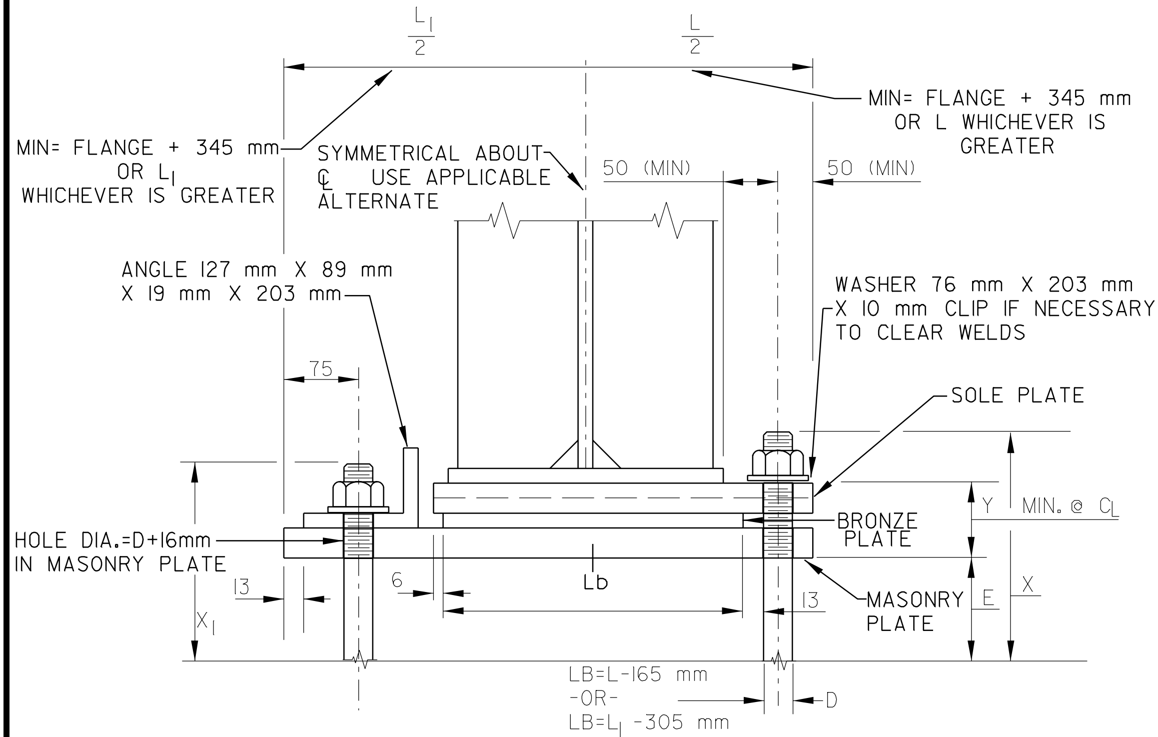


SLOTTED HOLE IN SOLE PLATE MAY BE DRILLED OR MACHINE BURNED.

SLOTS IN THE SOLE PLATE ARE:
1-⅞" X 2-⅞" FOR △ = 1"
2-⅛" X 3-⅝" FOR △ = 1"
2-⅛" X 4-⅛" FOR △ = 1-½"

NOTE: ALL DIMENSIONS ARE IN INCHES.

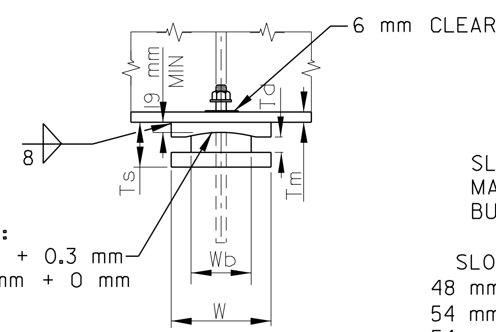
Figure 5-36b
Steel Bearings – Fixed [Metric]



ANCHOR BOLTS THROUGH SOLE PLATES ARE ONLY NECESSARY WHEN UPLIFT COULD OCCUR. WHEN BOLTS PASS THROUGH SOLE PLATES, EXTREME CARE SHALL BE EXERCISED IN PROPERLY LOCATING ANCHOR BOLTS.

kN	mm												
R	L	W	Wd	Ts	Tm	Tb	D	E	X	Y	△	I_1	X_1
535	508	254	203	44	51	38	32	305	476	108	25	647	438
800	559	305	241	51	57	38	32	305	483	114	25	700	444
980	610	267	254	57	63	44	38	381	572	127	38	750	527
1150	660	343	280	63	63	51	38	381	578	133	38	800	527
1335	711	356	280	63	70	51	38	381	584	139	51	851	533
1510	762	419	292	70	70	51	38	381	584	139	51	902	533
1780	914	394	317	76	76	57	38	381	597	152	51	953	540

NOTE:
SEE FIGURE 5-29



381mm RADIUS TOLERANCES:
SOLE PLATE 381mm -0 mm + 0.3 mm
BRONZE PLATE 381mm -3 mm + 0 mm

SLOTTED HOLE IN SOLE PLATE MAY BE DRILLED OR MACHINE BURNED.

SLOTS IN THE SOLE PLATE ARE:
48 mm X 73 mm FOR △ = 25 mm
54 mm X 92 mm FOR △ = 38 mm
54 mm X 105 mm FOR △ = 51mm

NOTE: ALL DIMENSIONS ARE IN MILLIMETERS.

Figure 5-37a
Expansion High-Load Multi-Rotational Bearing Detail Example

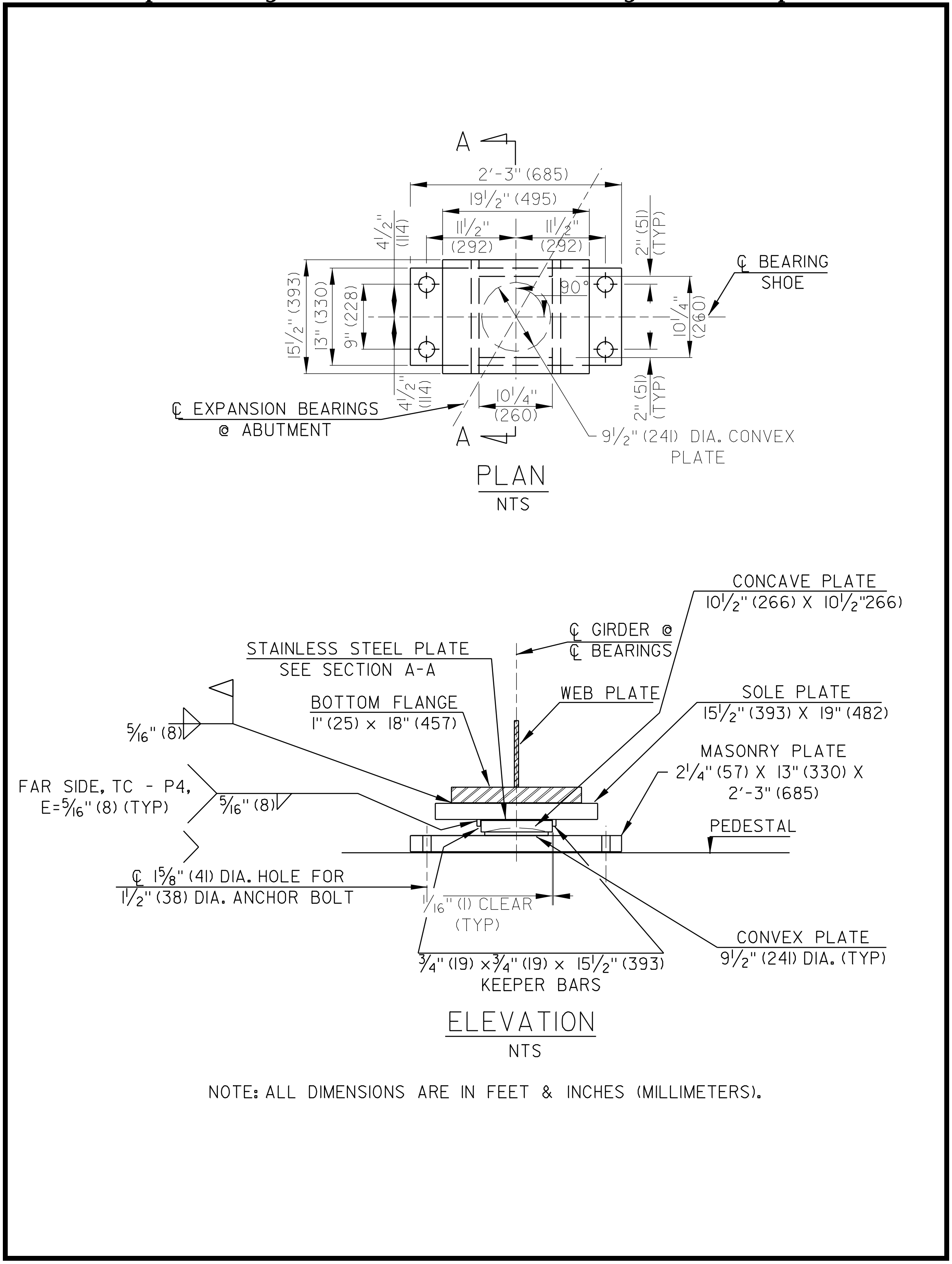


Figure 5-37b
Expansion High-Load Multi-Rotational Bearing Detail Example

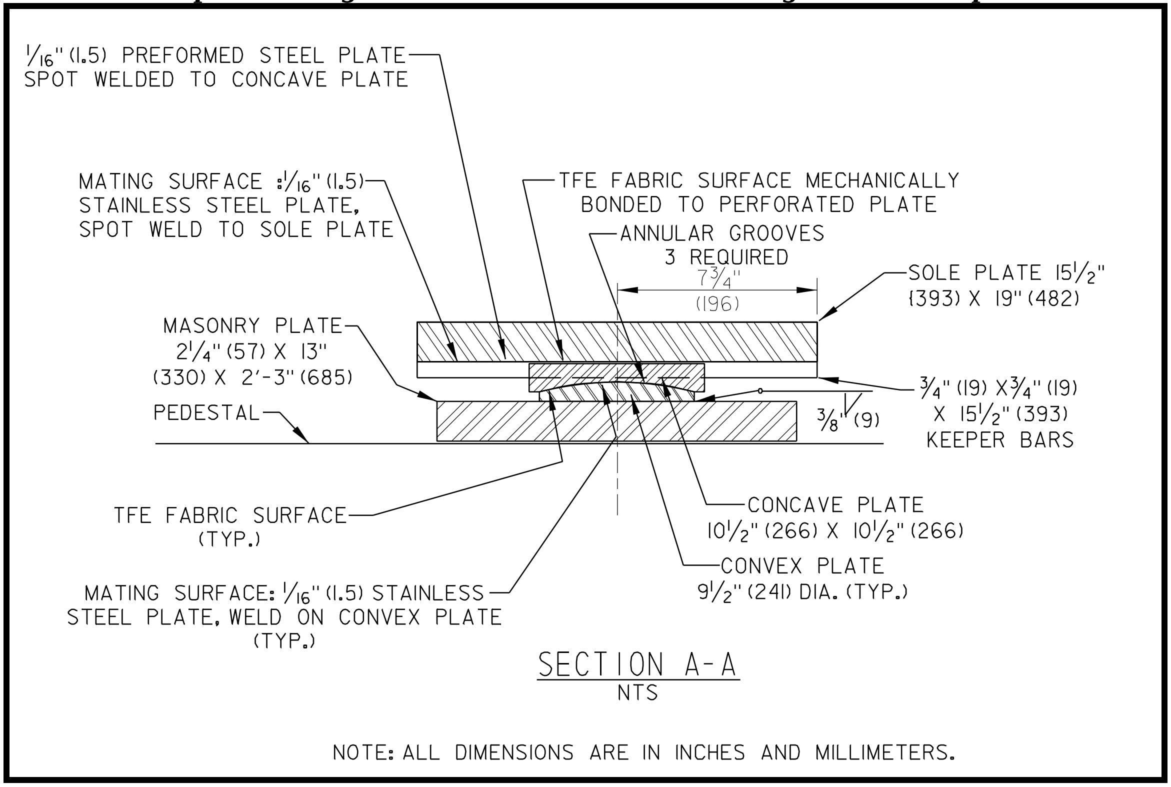


Figure 5-37c
Fixed High-Load Multi-Rotational Bearing Detail Example

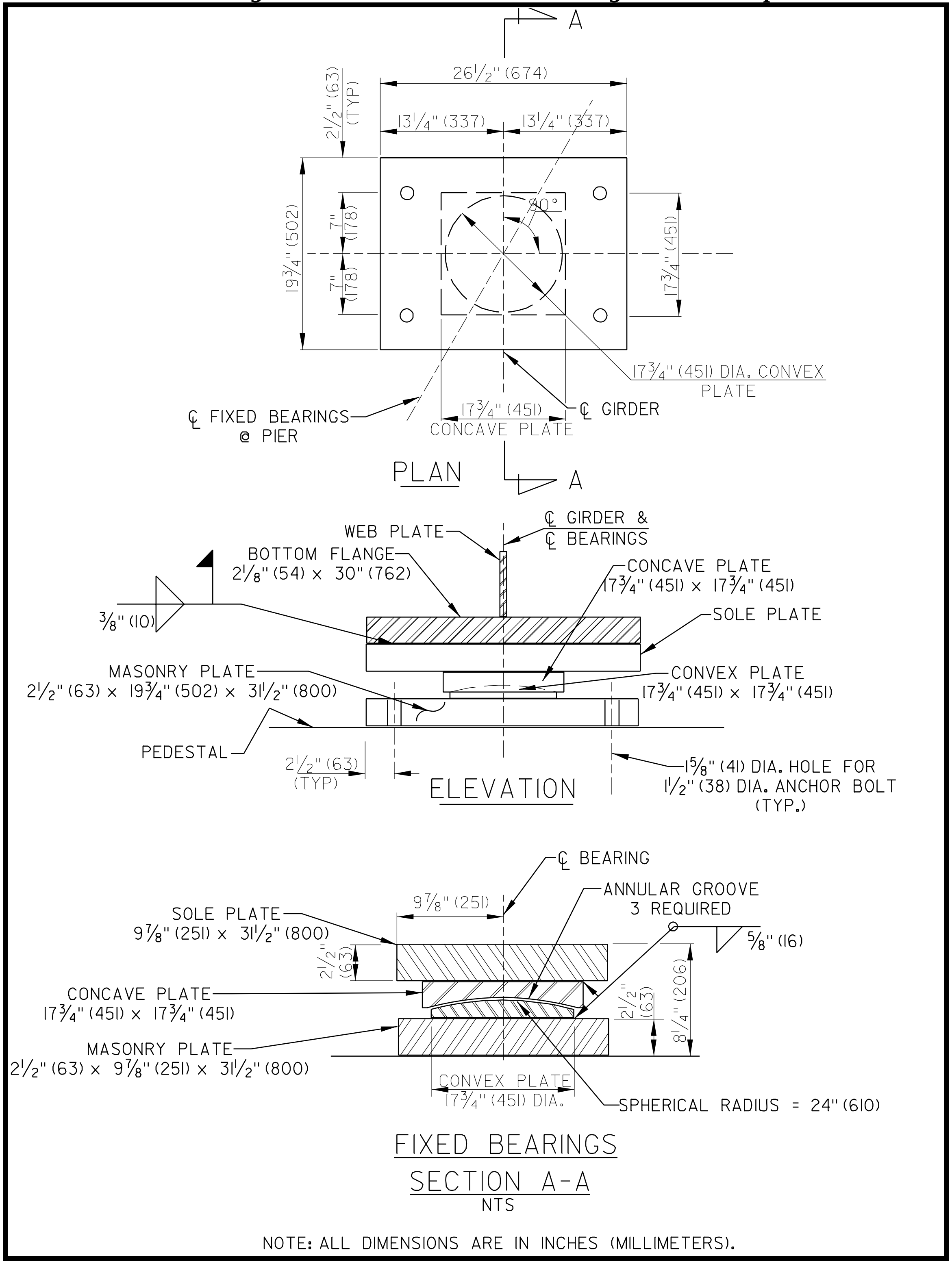
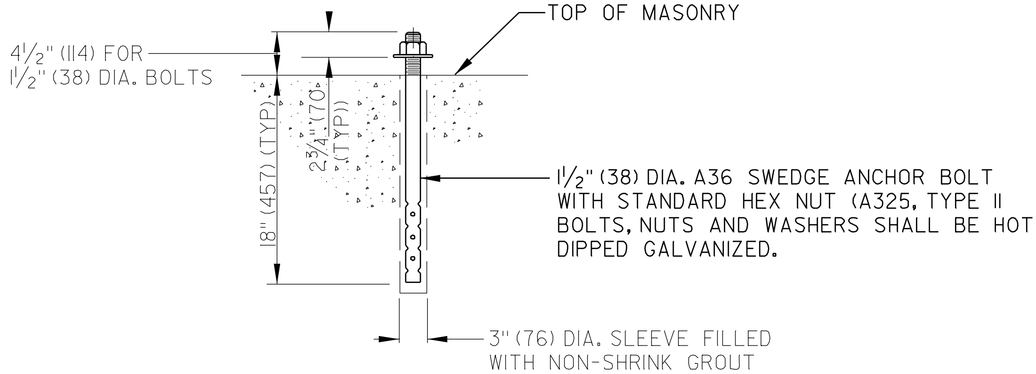


Figure 5-37d
High-Load Multi-Rotational Bearing Detail Example

BEARING NOTES:

1. BEARINGS SHALL PERMIT A ROTATION OF 2 DEGREES IN ALL DIRECTIONS.
2. STEEL FOR THE MASONRY PLATE, SOLE PLATE, CONVEX PLATE, AND CONCAVE PLATE SHALL MEET THE REQUIREMENTS OF AASHTO DESIGNATION M270, GRADE 50 (345). STEEL SURFACES NOT IN CONTACT WITH ONE ANOTHER OR WITH TFE FABRIC SURFACE SHALL BE PAINTED.

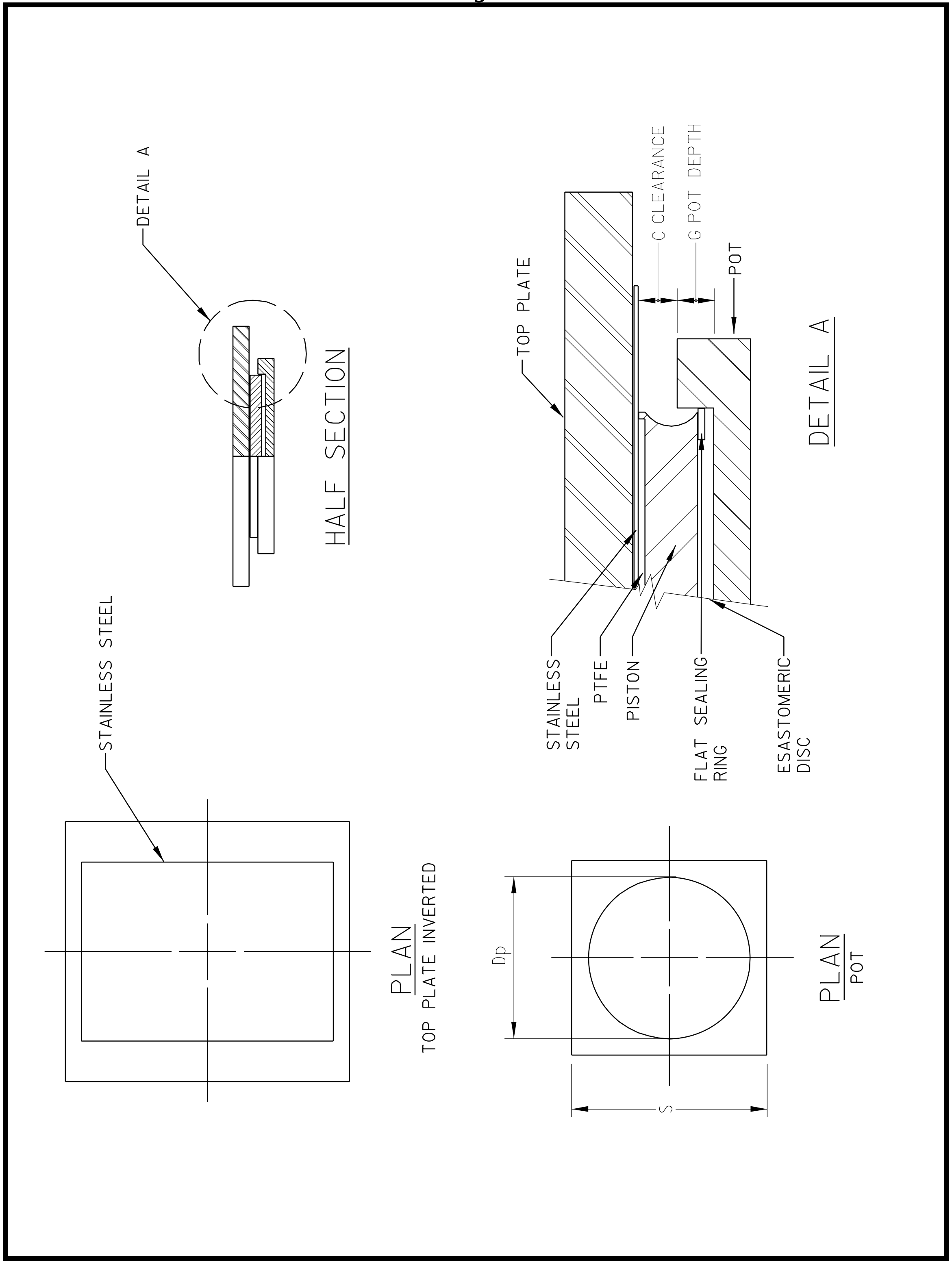


ANCHOR BOLT DETAIL

NTS

NOTE: ALL DIMENSIONS ARE IN INCHES AND MILLIMETERS.

Figure 5-38
Pot Bearing Details



For pot or rotational bearing design, refer to FHWA *Mid-Atlantic States SCEF Standard 106, High Load Multi-Rotational Bearings*. The design requirements for bearings are the same for either steel or concrete beams.

For anchor bolt design, refer to Section 14.8.3, Anchorage and Anchor Bolts, in the *AASHTO Specifications*. Anchor bolts must meet seismic requirements.

5.6.2 SEISMIC PROVISIONS

The United States is divided into four Seismic Performance Zones (SPZ) (1 through 4) based on the acceleration coefficient and the importance category. Areas in SPZ "1" are the least likely to experience earthquakes. Refer to Section 3.10, Earthquake Effects, and Section 4.7.4, Analysis for Earthquake Loads, in the *AASHTO Specifications*. Seismic provisions apply to the connections between the superstructure and substructure.

All of Delaware is in either SPZ "1" or "2." The boundary is Route 273. All bridges on, over or north of Route 273 belong to SPZ "2."

The design requirements for SPZ "1" are given in Section 3.10.9.2, Seismic Zone 1, in the *AASHTO Specifications*. The design requirements for SPZ "2" are more complex and are given in Section 3.10.9.3, Seismic Zone 2, in the *AASHTO Specifications.*

5.7 SLAB BRIDGES

Generally, slab bridge designs are not used for new bridges. They should be considered for locations where other bridge types cannot meet the required vertical clearance.

5.7.1 MATERIALS

All concrete for precast slab bridge decks will meet f'_c equals 5,000 psi [35 MPa]. All other concrete will meet f'_c equals 4,500 psi [30 MPa]. Higher strength concrete may be considered with approval by the Bridge Design Engineer.

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified. The minimum size of reinforcing bar is #5 [16] bar.

All reinforcing steel shall be protected with fusion-bonded epoxy. Epoxy coating conforming with AASHTO M284 [M284M] shall be specified.

5.7.2 DESIGN

Refer to Section 3, Loads and Load Factors, and Section 5, Concrete Structures, in the *AASHTO Specifications* for design requirements.

Usually, slab bridges are used for short spans, 20 feet [6 m] or less. No provisions for expansion or contraction are needed for one-span slab bridges. Provisions for expansion and contraction are required for simple and continuous, multi-span structures.

Voided slabs may be used where reduced superstructure weight is needed. Drains must be provided for each void.

5.7.3 THICKNESS

The minimum thickness of concrete for slab bridge decks is 10 in [250 mm].

5.7.4 CONCRETE COVER

The minimum cover over the rebar is:

- Top—2.5 in [65 mm], including a 0.5 inch [15 mm] integral wearing surface;
- Bottom—2 in [50 mm].

5.8 APPROACH SLABS

Generally, fill placed behind abutments settles after the bridge is opened to traffic. For this reason, the Department's policy is to construct reinforced concrete approach slabs to span the fill area in the following cases:

- all interstate routes and freeways; and
- other high-volume, high-speed routes.

Approach slabs are not normally used on low-volume, low-speed roads. Settlement of approach roadway embankments may also be addressed by methods covered in Section 6.7.

5.8.1 MATERIALS

All concrete for approach slabs shall be Class D and meet f'_c equals 4,500 psi [30 MPa].

Reinforcing steel meeting the requirements for AASHTO M31, Grade 60 [M31M, Grade 420], shall be specified. The minimum size of reinforcing bar is #5 [16] bar.

All reinforcing steel shall be protected with fusion-bonded epoxy. Epoxy coating conforming with AASHTO M284 [M284M] shall be specified.

5.8.2 DESIGN

Refer to Section 5, Concrete Structures, in the *AASHTO Specifications* for design criteria. Approach slabs are designed as simple spans, assuming no support from the abutment backfill. The length of approach slabs varies depending on the height of the abutments. The span length used for design of the approach slab is determined as shown in Figure 5-39.

Refer to Section 5.14.4, Slab Superstructures, *AASHTO Specifications,* for design of edge beams for approach slabs. More refined methods of analysis may be used for approach slab design as approved by the Bridge Design Engineer.

5.8.3 THICKNESS

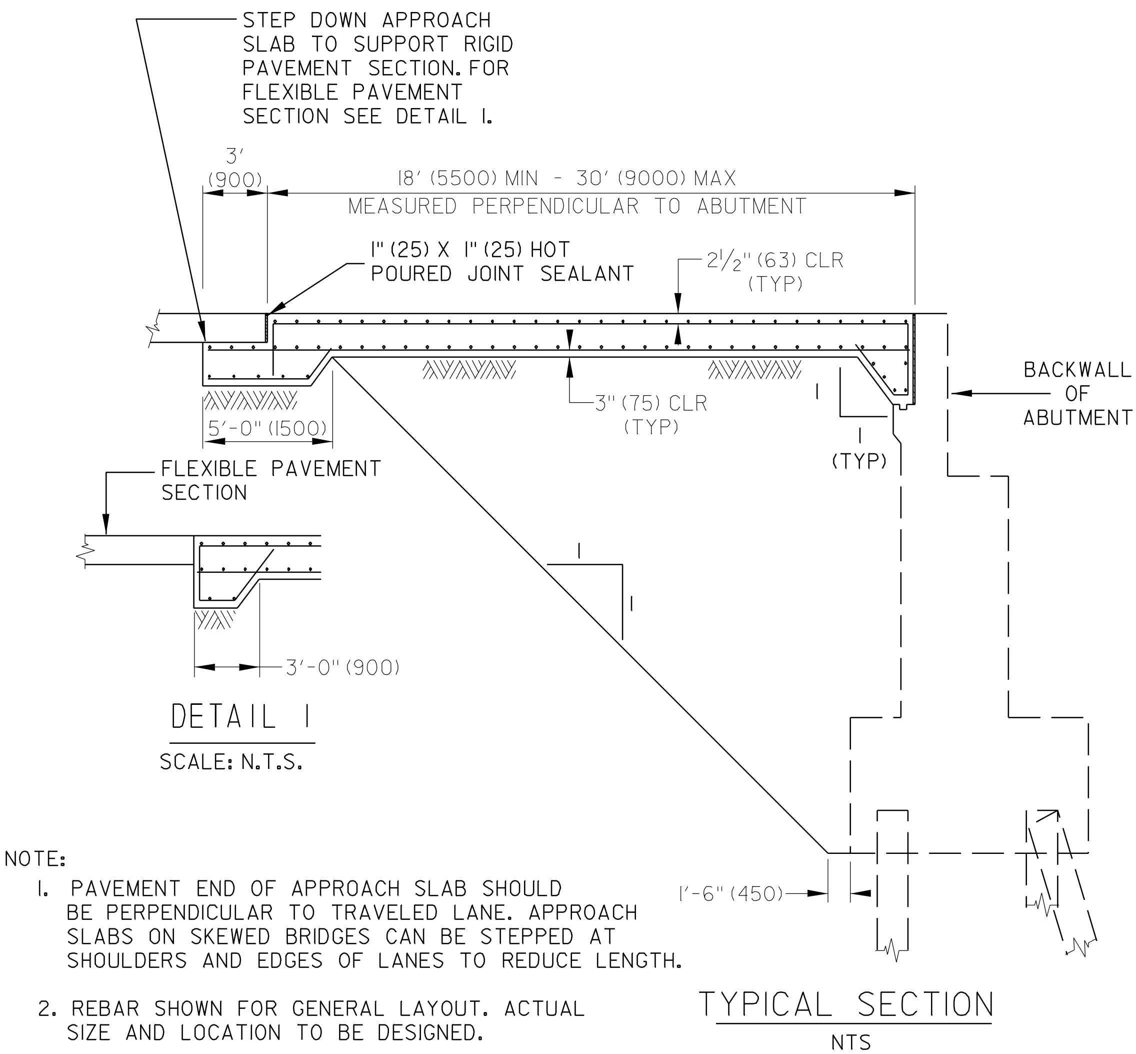
The slab thickness will be determined by the design. There is no minimum thickness.

5.8.4 CONCRETE COVER

The minimum concrete cover over reinforcing steel in approach slabs is:

- Top — 2.5 in [65 mm], and
- Bottom — 3 in [75 mm], as with other unformed concrete in contact with the ground.

Figure 5-39a
Approach Slab Design Details



NOTE:

1. PAVEMENT END OF APPROACH SLAB SHOULD BE PERPENDICULAR TO TRAVELED LANE. APPROACH SLABS ON SKEWED BRIDGES CAN BE STEPPED AT SHOULDERS AND EDGES OF LANES TO REDUCE LENGTH.
2. REBAR SHOWN FOR GENERAL LAYOUT. ACTUAL SIZE AND LOCATION TO BE DESIGNED.
3. ALL BARS TO BE EPOXY COATED.

CASE A - AS SHOWN, WITH EXISTING GROUND AT OR ABOVE THE BOTTOM OF FOOTING.

CASE B - FOR EXISTING GROUND BELOW BOTTOM OF FOOTING. THE ENGINEER SHALL DETERMINE THE REQUIRED LENGTH OF APPROACH SLAB. USING THE ABOVE CRITERIA AS MINIMUM WITH A 30′ (9m) MAXIMUM.

NOTE: ALL DIMENSIONS ARE IN INCHES AND MILLIMETERS.

Figure 5-39b
Approach Slab Design Details

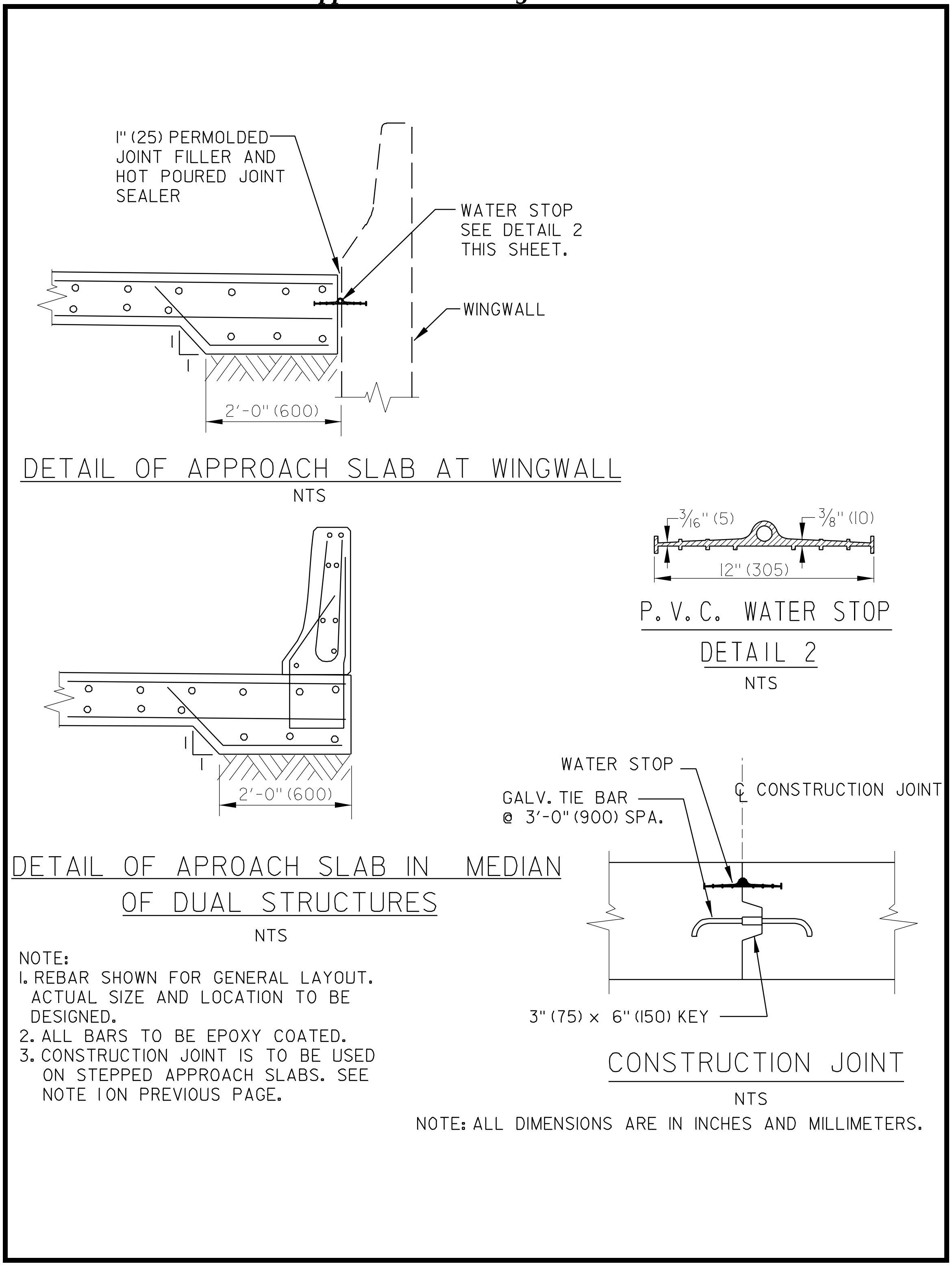


Figure 5-39c
Approach Slab Design Details